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PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

CONTENTS

CONTENTS (*Continued*)

Discussions (*Continued*):

Memoirs:

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY DISTRIBUTING FIXED-END MOMENTS

By Hardy Cross,* M. Am. Soc. C. E.

Synopsis

The purpose of this paper is to explain briefly a method which has been found useful in analyzing frames which are statistically indeterminate. The essential idea which the writer wishes to present involves no mathematical relations except the simplest arithmetic. It is true that in order to apply the method it is necessary to determine certain constants mathematically, but the means to be used in determining these constants are not discussed in the paper, nor are they a part of the method. These constants have been derived by so many writers and in so many slightly different ways that there is little occasion to repeat here the whole procedure.

The reactions in beams, bents, and arches which are immovably fixed at their ends have been extensively discussed. They can be found comparatively readily by methods which are more or less standard. The method of analysis herein presented enables one to derive from these the moments, shears, and thrusts required in the design of complicated continuous frames.

Definitions

For convenience of reference, definitions of three terms will be introduced at once. These terms are "fixed-end moment", "stiffness", and "carry-over factor".

By "fixed-end moment" in a member is meant the moment which would exist at the ends of the member if its ends were fixed against rotation.

"Stiffness", as herein used, is the moment at one end of a member (which is on unyielding supports at both ends) necessary to produce unit rotation of that end when the other end is fixed.

If one end of a member which is on unyielding supports at both ends is rotated while the other end is held fixed the ratio of the moment at the fixed

Note.—Written discussion on this paper will be closed in September, 1930, *Proceedings*.

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end to the moment producing rotation at the rotating end is herein called the "carry-over factor."

Effect of Joint Rotation

Imagine any joint in a structure, the members of which are being deformed by loads, or in some other way, to be first held against rotation and then released. Call the algebraic sum of the fixed-end moments at the joint the "unbalanced fixed-end moment". Before the joint is released this unbalanced fixed-end moment will not usually be zero; after the joint is released, the sum of the end moments at the joint must be zero. The total change in end moments, then, must equal the unbalanced fixed-end moment. This may be stated in another way by saying that the unbalanced fixed-end moment has been "distributed to" the connecting members in some ratio.

When the joint is released all connecting members rotate through the same angle and this rotation at the end is accompanied by a change in end moment. The change in end moments is proportional to the "stiffness" of the members.

Hence, it may be said that when the joint is released the unbalanced fixed-end moment is distributed among the connecting members in proportion to their stiffness.

The rotation of the joint to produce equilibrium induces moments at the other ends of the connecting members. These are equal in each member to the moments distributed at the rotating joint multiplied by the carry-over factor at the rotating end of the member. This follows from the definition of "carry-over factor".

Moment Distribution

The method of moment distribution is this: (*a*) Imagine all joints in the structure held so that they cannot rotate. Compute the moments at the ends of the members for this condition; (*b*) at each joint distribute the unbalanced fixed-end moment among the connecting members in proportion to the constant for each member defined as "stiffness"; (*c*) multiply the moment distributed to each member at a joint by the carry-over factor at that end of the member and set this product at the other end of the member; (*d*) distribute these moments just "carried over"; (*e*) repeat the process until the moments to be carried over are small enough to be neglected; and (*f*) add all moments —fixed-end moments, distributed moments, moments carried over—at each end of each member to obtain the true moment at the end.

To the mathematically inclined the method will appear as one of solving a series of normal simultaneous equations by successive approximation. From an engineering viewpoint it seems simpler and more useful to think of the solution as if it were a physical occurrence. The beams are loaded or otherwise distorted while the joints are held against rotation; one joint is then allowed to rotate with accompanying distribution of the unbalanced moment at that joint and the resulting moments are carried over to the adjacent joints; then another joint is allowed to rotate while the others are held against rotation; and the process is repeated until all the joints are "eased down" into equilibrium.

Beam Constants

This method of analysis is dependent on the solution of three problems in the mechanics of materials. These are the determination of the fixed-end moments, of the stiffness at each end, and of the carry-over factor at each end for each member of the frame under consideration. The determination of these values is not a part of the method of moment distribution and is not discussed in this paper.

The stiffness of a beam of constant section is proportional to the moment of inertia divided by the span length, and the carry-over factor is $-\frac{1}{2}$.

The proof or derivation of these two statements and the derivation of formulas for fixed-end moments is left to the reader. They can be deduced by the use of the calculus; by the theorems of area-moments; from relations stated in *Bulletin 108* of the Engineering Experiment Station of the University of Illinois (the Slope-Deflection *Bulletin*); from the theorem of three moments; by what is known to some as the column analogy method;* or by any of the other corollaries of geometry as applied to a bent member. Formulas for fixed-end moments in beams of uniform section may be found in any structural handbook.

Signs of the Bending Moments

It has seemed to the writer very important to maintain the usual and familiar conventions for signs of bending moments, since these are the conventions used in design.

For girders the usual convention is used, positive moment being such as sags the beam. For vertical members the same convention is applicable as for girders if the sheet is turned to read from the right as vertical members on a drawing are usually read. The usual conventions for bending moments are, then, applicable to both girders and columns if they are looked at as a drawing is usually lettered and read.

Moments at the top of a column, as the column stands in the structure, should be written above the column and those at the bottom of the column, as the column stands in the structure, should be written below the column when the sheet is in position to read the columns. This is necessary because positive moment at the right end of a beam and at the top of a column both represent tendencies to rotate the connected joint in the clockwise direction.

It makes no difference whether girder moments are written above or below the girder. Either arrangement may be convenient. Confusion will be avoided by writing column moments parallel to the column and girder moments parallel to the girders.

When any joint is balanced the total moment to the right and to the left of the support is the same, both in absolute value and in sign. The unbalanced moment is the algebraic difference of the moments on the two sides of the joint.

Limitation of Method

From the fact that the terms, "stiffness" and "carry-over factor", have been defined for beams resting on unyielding supports, it follows that direct

* "The Column Analogy," by Hardy Cross, M. Am. Soc. C. E., *Bulletin 203*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill. (In press.)

application of the method is restricted to those cases where the joints do not move during the process of moment distribution. The method, however, can be applied in an indirect way to cases in which the joints are displaced during the moment distribution, as indicated later.

As the method has been stated, it is restricted only by this condition that the joints are not displaced. If this condition is satisfied it makes no difference whether the members are of constant or of varying section, curved or straight, provided the constants, (*a*) fixed-end moments at each end, (*b*) stiffness at each end, and (*c*) carry-over factor at each end, are known or can be determined. Such values can be derived by standard methods and may be tabulated for different types of members and conditions of loading.

It will be found that in most cases accuracy is needed only in the fixed-end moments. It does not ordinarily make very much difference how, within reason, the unbalanced moments are distributed, nor, within reason, how much of the distributed moments are carried over.

In the illustration which follows it has been assumed that the members are straight and of uniform section. The stiffnesses, then, are proportional to the moments of inertia, (I), divided by the lengths, (L), but the relative values given for $\frac{I}{L}$ in this problem might quite as well be the relative stiffness of a series of beams of varying section. In this latter case, however, the carry-over factors for the beams would not be $-\frac{1}{2}$.

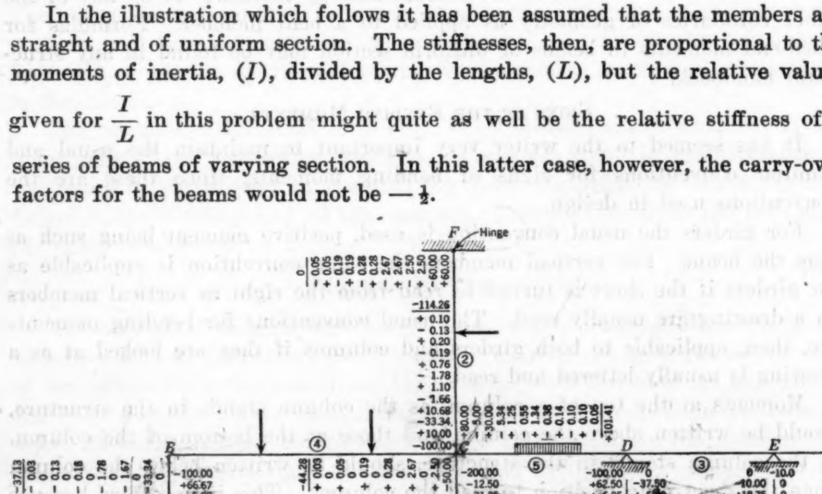

FIG. 1.

ILLUSTRATION

The illustration given (Fig. 1) is entirely academic. It is not intended to represent any particular type of structure nor any probable condition of loading. It has the advantage for the purpose of this paper that it involves all the conditions that can occur in a frame which is made up of straight members and in which the joints are not displaced.

The loads on the frame are supposed to be as indicated. The relative values of $\frac{I}{L}$ for the different members are indicated in circles.

The fixed-end moments in all members are first written. In this problem they are arbitrarily assumed to be as shown, as follows: at A, 0; at B, in $B A$, 0, and in $B C$, — 100; at C, in $C B$, — 100, in $C F$, + 80, in $C D$, — 200, and in $C G$, — 50; at F, + 60; at G, — 50; at D, in $D C$, — 100, and in $D E$, 0; at E, in $E D$, 0, and in the cantilever, — 10.

Before proceeding to a solution of the problem, attention may be called to the arrangement of the computations. The moments in the girders are written parallel to the girders; those in the columns, parallel to the columns. The original fixed-end moments are written next to the members in which they occur, the successive moments distributed or carried over being written above or below these, but farther from the member.

The arrangement of the moments in the columns in positions above the columns, when the paper is turned into a position to write these moments, for the top of the columns (at B, F, and C), and in positions below the columns for the bottom of the columns (at A, C, and G), is an essential part of the sign convention adopted.

The moment at C in the girder, $B C$, is written above the girder in order to get it out of the way. Otherwise, it makes no difference whether the moments are written above or below the girder.

The signs of the fixed-end moments are determined by observing the direction of flexure at the ends of the members due to the loads. In order to apply to the columns the ordinary conventions for signs of bending moments it is necessary to turn the drawing of the structure.

The reader should realize that the solution is built up step by step. It is always the last figures showing that are to be operated on—distributed or carried over—so that in ordinary framework there is little chance for confusion as to what step should be taken next.

Distribute at each joint the unbalanced moment, as follows:

1.—At A there is no moment.

2.—At B there is an unbalanced moment of — 100 on one side of the joint. This moment is distributed to $B A$ and to $B C$ in the ratio, 2 : 4, so that the distributed moment to $B A$ is $\frac{2}{2+4} 100 = 33.33$ and to $B C$, $\frac{4}{2+4} 100 = 66.67$. The signs are written in the only way possible to balance the joint by giving the same total moment (— 33.33) both to left and right of the joint.

3.—At C, the unbalanced moments are, in $C B$, — 100, and in $C G$, — 50, giving a total of — 150 on the left of the joint; in $C F$, + 80, and in $C D$, — 200, giving a total of — 120 on the right of the joint. The total unbalanced moment at the joint, which is the difference between the total moment on the left and on the right of the joint, is 30. This is now distributed in the respective proportions, as follows:

To $C\,B$,

$$\frac{4}{4+2+5+1}\,30 = 10$$

to $C\,F$,

$$\frac{2}{4+2+5+1}\,30 = 5$$

to $C\,D$,

$$\frac{5}{4+2+5+1}\,30 = 12.5$$

and, to $C\,G$,

$$\frac{1}{4+2+5+1}\,30 = 2.5$$

There is only one way to place the signs of the distributed moments so that the total is the same on both sides of the joint. This is done by reducing the excess negative moment on the left and increasing the negative moment on the right.

4.—At F, the unbalanced moment is $+ 60$. The hinge has no stiffness. The moment, then, is distributed between the member, $F\,C$, and the hinge in the ratio, 2 : 0; all of it goes to the member. The total balanced moment is $+ 60 - 60 = 0$, as it must be at a free end.

5.—At G, the abutment is infinitely stiff and the unbalanced moment, $- 50$, is distributed between the member, $G\,C$, and the abutment in the ratio, $1 : \infty$. The member gets none of it; the end stays fixed.

6.—At D, the unbalanced moment, $- 100$, is distributed to $D\,C$ and to $D\,E$ in the ratio of 5 : 3.

7.—At E, the unbalanced moment is $- 10$ in the cantilever. Since the cantilever has no stiffness, this unbalanced moment is distributed between the beam, $E\,D$, and the cantilever in the ratio, 3 : 0. This means that all of it goes to $E\,D$.

All joints have now been balanced. Next, carry over from each end of each member one-half the distributed moment just written, reverse the sign, and write it at the other end of the member. Thus, carry over, successively, in $A\,B$, 0 from A to B and $+ 16.67$ from B to A; in $B\,C$, $- 33.34$ from B to C and $- 5.0$ from C to B; in $C\,F$, $+ 2.5$ from C to F and $+ 30$ from F to C; in $C\,G$, 0 from G to C and $- 1.25$ from C to G; in $C\,D$, $+ 6.25$ from C to D and $- 31.25$ from D to C; and in $D\,E$, $+ 18.75$ from D to E and $+ 5.00$ from E to D.

Distribute the moments just carried over exactly as the original fixed-end moments were distributed. Thus, at A, $+ 16.67$ is distributed 0 to $A\,B$ (fixed-ended); at B, $- 5.0$ is distributed as $- 1.67$ and $+ 3.32$; at C, the unbalanced moment is $(- 33.34 + 0) - (+ 30.00 - 31.25) = - 32.09$ which is distributed as $+ 2.67$, $+ 10.68$, $- 5.34$, and $- 13.35$; at F, $+ 2.50$ is distributed as $- 2.5$ to the member; G is fixed-ended; at D, $+ 1.25$ is distributed as $- 0.78$ and $+ 0.47$; at E, the unbalanced $+ 18.75$ is distributed to the member as $- 18.75$.

The moments distributed are now carried over as before and then re-distributed; and the process is repeated as often as desired. The procedure should

be stopped after each distribution, however, and a check made to see that statics ($\Sigma M = 0$) is satisfied.

When it is felt that the process has gone far enough, all moments at each end of each member are added to give the total moment at the joint. After the moments at the joints have been determined, all other quantities, such as moments and shears, may be obtained by applying the laws of statics.

CONVERGENCE OF RESULTS

The distribution herein has been carried out with more precision than is ordinarily necessary, in order to show the convergence of the results. To show the rate of convergence, the successive values of the moments at the joints after successive distributions are given in Table 1.

TABLE 1.—CONVERGENCE OF RESULTS.

Successive values of bending moment at joint.		After one distribution (two rows of figures).	After two distributions (four rows of figures).	After three distributions (six rows of figures).	After four distributions (eight rows of figures).	After five distributions (ten rows of figures).	After six distributions (twelve rows of figures).
A	....	0	+ 16.67	+ 17.50	+ 18.39	+ 18.48	+ 18.55
B	....	− 33.34	− 35.01	− 36.79	− 36.97	− 37.10	− 37.13
C	In *CB*	− 90.00	−112.66	−113.22	−114.24	−114.23	−114.26
	" *CF*	+ 75.00	+ 99.66	+100.36	+101.82	+101.36	+101.41
	" *CD*	−212.50	−257.10	−258.09	−259.89	−259.88	−259.93
	" *CG*	− 47.50	− 44.83	− 44.55	− 44.36	− 44.31	− 44.28
D	....	− 37.50	− 32.03	− 23.66	− 23.48	− 23.15	− 23.15
E	....	− 10.00	− 10.00	− 10.00	− 10.00	− 10.00	− 10.00
F	....	0	0	0	0	0	0
G	....	− 50.00	− 51.25	− 52.59	− 52.73	− 52.83	− 52.86

For most purposes the computations might as well have been stopped after the second distribution. Had this been done, the solution would have appeared as shown in Fig. 2.

For any practical purpose the computation might in this case have been stopped after the third distribution. In general, two or three distributions are sufficient. This is not true in all instances, but in any case the exactness of the solution at any stage will be indicated by the magnitude of the moments carried over in the members.

VARIATIONS OF THE METHOD

The writer has developed and used at different times several variations of the method shown, but the original method is itself so simple and so easy to remember that he finds himself inclined to discard the variants.

One variant is perhaps worth recording. It is rather tedious to carry moments out to the end of a member which is free to rotate and then balance the moment and carry it back again. This may be avoided by releasing the free end once for all and leaving it free. In this case, for beams of constant section, the stiffness of the beam is to be taken three-fourths as great* as

* The moment needed to produce a given rotation at one end of a beam when the other end is free is three-fourths as great as if the other end is fixed.

the relative $\frac{I}{L}$-value would indicate. After the end of the beam is once released, no moments are carried over to it.

Correcting for Side-Sway

Single square or trapezoidal frames, portals, L-frames, box culverts, and similar structures act as simple continuous beams if there is no transverse deflection. If they are symmetrical as to form and loading, they will not deflect sidewise and if they are restrained against sidewise movement, they cannot so deflect.

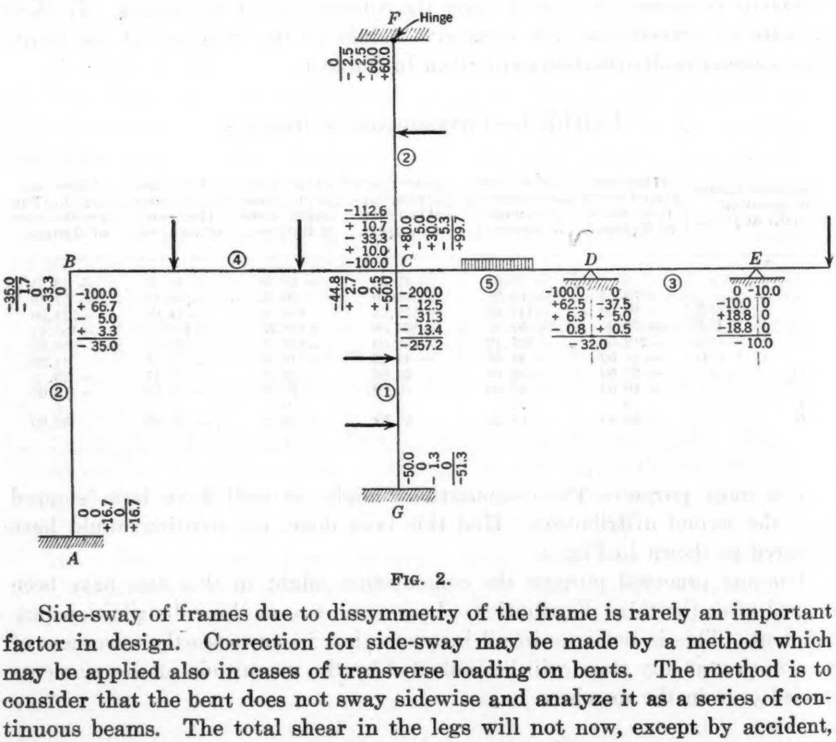

Fig. 2.

Side-sway of frames due to dissymmetry of the frame is rarely an important factor in design. Correction for side-sway may be made by a method which may be applied also in cases of transverse loading on bents. The method is to consider that the bent does not sway sidewise and analyze it as a series of continuous beams. The total shear in the legs will not now, except by accident, equal the shear which is known to exist. The difference must be a force which prevents side-sway.

Now, assume all joints held against rotation, but the top of the beam moved sidewise. Assume any series of fixed-end moments in the legs such that all legs have the same deflection. In this case for members of uniform section fixed-end moments in columns vary as $\frac{I}{L^2}$. Distribute these fixed-end moments and find the total shear in the legs. The changes in moments due to side-sway will then be to the moments just computed in the same algebraic ratio as the total unbalanced horizontal shear in the legs due to side-sway, when the frame is analyzed as a continuous girder, is to the shear just computed.

Multi-Storied Bents

Bents of more than one story, subject to side-sway, either as a result of unbalanced loading or due to horizontal forces, may be solved by this method. It is understood that exact solution of such problems is not commonly of great interest. It is the approximate effect that is desired rather than exact analysis.

To analyze by this method a two-story bent it will be necessary to make two configurations—one for each story. From the assumed shear in each story (producing, of course, shears in the other stories), a set of moment values may be obtained. These may be combined to obtain the true shears, and from the true shears the true moments follow.

General Application of the Method

The method herein indicated of distributing unbalanced moments may be extended to include unbalanced joint forces. As thus extended it has very wide application. Horizontal or vertical reactions may be distributed and carried over and thus a quick estimate made of the effect of many complicating elements in design. The writer has used it in studying such problems as continuous arch series, the effect of the deflection of supporting girders, and other phenomena.

An obvious application of moment distribution occurs in the computation of secondary stresses in trusses. Many other applications will doubtless suggest themselves, but it has been thought best to restrict this paper chiefly to continuous frames in which the joints do not move.

Conclusion

The paper has been confined to a method of analysis, because it has seemed wiser to so restrict it. It is not then an oversight that it does not deal with: (1) Methods of constructing curves of maximum moments; (2) methods of constructing curves of maximum shears; (3) the importance of analyses for continuity in the design of concrete girders; (4) flexural stresses in concrete columns; (5) methods of constructing influence lines; (6) the degree to which continuity exists in ordinary steel frames; (7) continuity in welded steel frames; (8) plastic deformation beyond the yield point as an element in interpreting secondary stress computations; (9) the effect of time yield on moments and shears in continuous concrete frames; (10) plastic flow of concrete as a factor in the design of continuous concrete frames; (11) whether in concrete frames it is better to guess at the moments, to take results from studies made by Winkler fifty years ago, or to compute them; (12) the effect of torsion of connecting members; (13) the relative economy of continuous structures; (14) the relative flexibility of continuous structures; (15) the application of methods of continuous frame analysis to the design of flat slabs; (16) probaability of loading and reversal of stress as factors in the design of continuous frames; (17) the relation of precision in the determination of shears and moments to precision in the determination of fiber stresses, and a dozen other considerations bearing on the design of continuous frames.

The writer has discussed several of these questions elsewhere. He hopes that readers will discuss some of them now.

A method of analysis has value if it is ultimately useful to the designer; not otherwise. There are apparently three schools of thought as to the value of analyses of continuous frames. Some say, "Since these problems cannot be solved with exactness because of physical uncertainties, why try to solve them at all?" Others say, "The values of the moments and shears cannot be found exactly; do not try to find them exactly; use a method of analysis which will combine reasonable precision with speed." Still others say, "It is best to be absolutely exact in the analysis and to introduce all elements of judgment after making the analysis."

The writer belongs to the second school; he respects but finds difficulty in understanding the viewpoint of the other two. Those who agree with his viewpoint will find the method herein explained a useful guide to judgment in design.

Members of the last named school of thought should note that the method here presented is absolutely exact if absolute exactness is desired. It is a method of successive approximations; not an approximate method.

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PAPERS AND DISCUSSIONS

THE TRAINING WALL ACROSS THE LIAO BAR IN MANCHURIA

By P. N. Fawcett,* M. Am. Soc. C. E.

Synopsis

This paper describes in detail the problems in harbor work encountered at the Port of Niuchuang in Southern Manchuria. The discharge of the Liao River has been such as to form a bar at its mouth that continually hampers free navigation. The paper presents a description of the movement of this bar before the construction of the single training wall as well as during its construction. It also contains supplementary data concerning tides, winds, rainfall, velocities of river flow, and other information important to a study of this kind.

Location

The Liao River is the most important waterway in Manchuria. Its length is 600 miles (of which 350 are navigable for junk traffic) and its drainage area is 80 000 sq. miles. It empties into the northern extremity of the shallow Gulf of Liaotung and the Gulf of Pechihli. The Port of Niuchuang, for which the improvements described in this paper were made, is shown in Fig. 1, at the northern end of the Gulf of Liaotung.

Tidal Data

Fig. 2 shows the tidal curves for this river. Spring tides range between 10 and 13 ft. and neap tides between 7 and 8 ft.; the diurnal inequality during the spring tides is sometimes as much as 4 ft., the variation being confined to the high waters, with practically none in the low waters. Owing to the peculiar formation of the entrance to the Gulf of Pechihli the tides are greatly influenced by winds. A northerly wind drives the water out of the Gulf, thus reducing the depths along the coasts, while a southerly wind forces the water into it, between Korea and the Shantung Peninsula, thereby aug-

Note.—Written discussion on this paper will be closed in **August, 1930**, *Proceedings*.
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menting the depth all over this shoal Gulf. Fig. 3 contains curves of simultaneous tidal lines and of high-water and low-water planes corresponding to Fig. 2. The readings are for stations along the river from about 6 miles below the Signal Station to a station at Uu Tai Tzu, about 65 miles up stream from Swan Island (see Fig. 4).

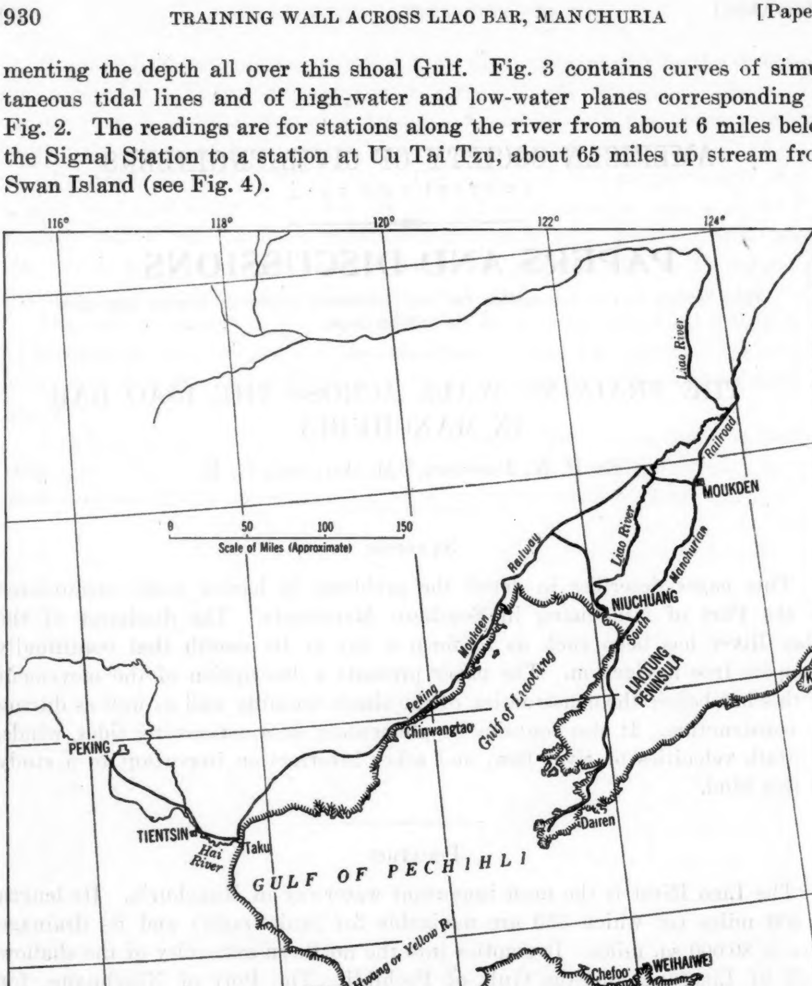

FIG. 1.—MAP OF DRAINAGE AREA OF THE LIAO RIVER.

The tidal charge of the flood of a spring tide, during the period of no freshets, is 140 000 000 cu. yd., or 210 000 cu. ft. per sec., and the ebb discharge is 152 000 000 cu. yd., or 154 000 cu. ft. per sec. The fresh-water discharge is 12 000 000 cu. yd., or less than one-tenth of the tidal charge. The current velocity past Niuchuang during spring flood tides is 4.8 knots, or 8 ft. per sec., and at ebb it is 3.06 knots, or 5 ft. per sec. During neap tides the current velocity of both flood and ebb tide past this point is 2.6 knots, or 4½ ft. per sec. These velocities do not obtain across the bar shown in Fig. 4. This

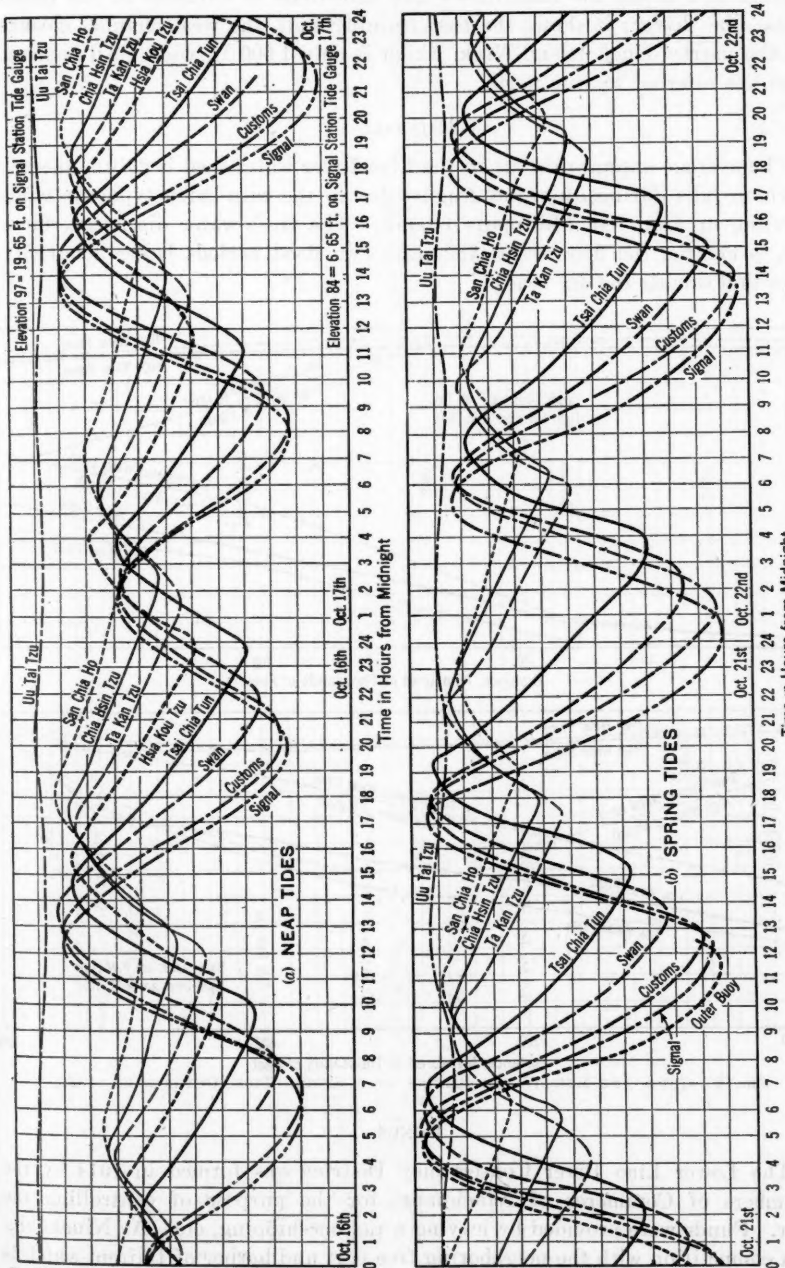

FIG. 2.—TIDE CURVES FOR THE LIAO RIVER.

diagram also shows the magnitudes and directions of currents at the mouth of the Liao River; it shows the east training wall, the west channel closure, and the narrow Duck Island Neck, which is only 1 600 ft. wide. All bearings given are referred to true north.

RAINFALL

The average annual rainfall on the Liao River water-shed is 23 in., one-half of which falls during July and August due to the rain-laden typhonic winds traveling up from the Philippine Islands. The fresh-water discharge, therefore, is erratic, the flow of the tributaries in flood periods being about fifty times that during the dry season.

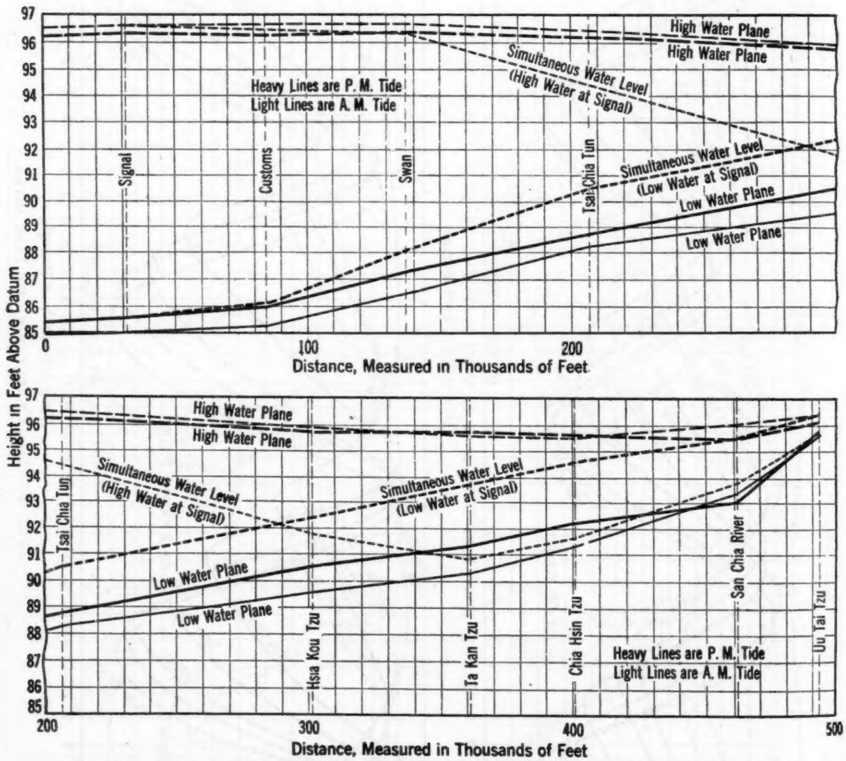

FIG. 3.—HIGH AND LOW-WATER PLANES OF A SPRING TIDE, OCTOBER 21, 1922.

FUNDS

The Lower Liao River Conservancy District was formed in 1914 by the Chambers of Commerce of Niuchuang, for the purpose of controlling the river. Funds were provided by levying a tax on shipping, etc. As Niuchuang is in competition with the neighboring free port and harbor of Dairen, which is owned and operated by the South Manchurian Railway Company, it follows that this surtax cannot be increased. If it were, the trade of Niuchuang,

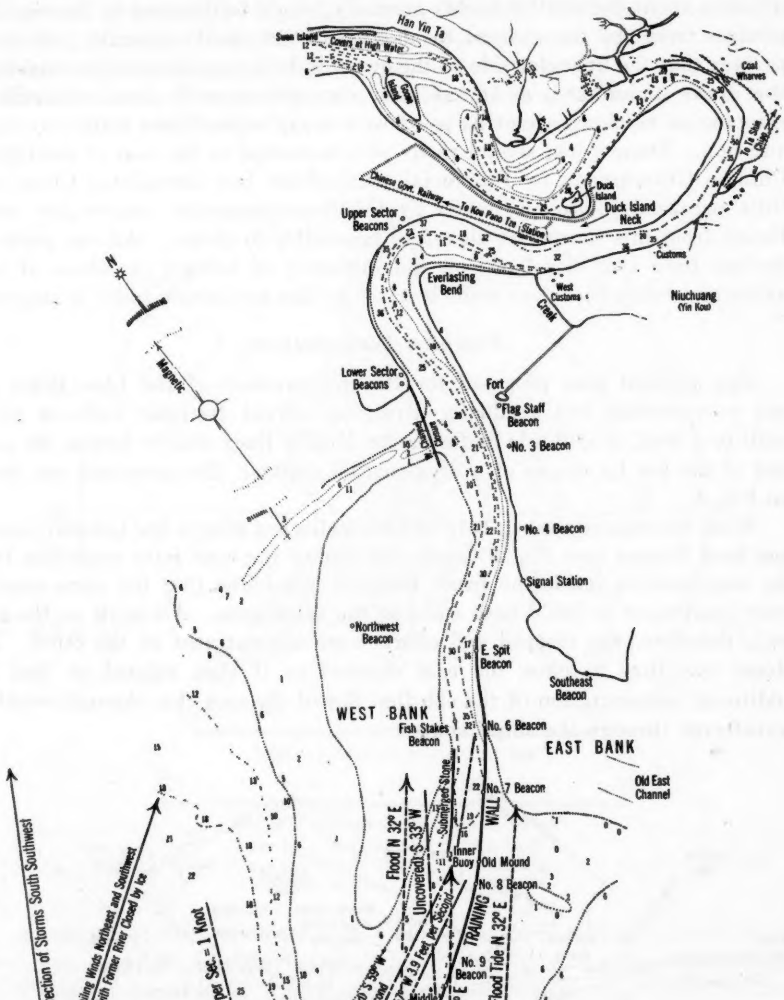

FIG. 4.—DIRECTIONS AND SPEEDS OF CURRENTS IN THE VICINITY OF LIAO BAR.

which is about \$50 000 000 (gold) annually, would be diverted to Dairen. The receipts from the tax amount to about \$135 000 (gold) annually. It is important that the financial side of the question be borne in mind in considering the works undertaken, as it has been very necessary to obtain benefits for shipping at as cheap a cost as possible, a heavy expenditure being out of the question. These works should really be constructed at the cost of the Central Chinese Government, or Provincial Authorities, but throughout China very little practical interest is taken by the Government in conservancy works. Funds from this source are therefore impossible to obtain. All the ports and harbors have been developed on the initiative of foreign chambers of commerce requesting that their trade be taxed so that navigation could be improved.

Proposed Improvements

The original plan proposed for the improvement of the Liao River Bar was to construct two gradually diverging curved training walls or jetties built to a level of mid-tide as far as the Middle Buoy and to deepen the outer end of the bar by means of a drag-suction dredge. The proposals are shown on Fig. 4.

Work was commenced in 1916 on both walls, but after a few isolated mounds had been formed (see Fig. 5) along the line of the west jetty extending from the west bank to inside the Inner Buoy, it was found that the stone supplies were insufficient to build both walls at the same time. All work on the west wall, therefore, was stopped and efforts were concentrated on the other. The object was thus to close the east channel as it then existed so that the additional concentration of the ebb that flowed through this channel would be transferred through the ships' course.

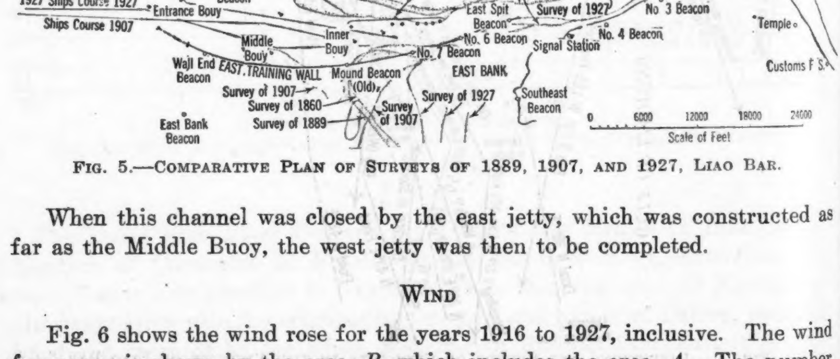

Fig. 5.—Comparative Plan of Surveys of 1889, 1907, and 1927, Liao Bar.

When this channel was closed by the east jetty, which was constructed as far as the Middle Buoy, the west jetty was then to be completed.

Wind

Fig. 6 shows the wind rose for the years 1916 to 1927, inclusive. The wind frequency is shown by the area, *B*, which includes the area, *A*. The number

of hours per year that the wind blew from different directions is obtained by measuring, with the frequency scale, the radial distance from the center of the circle to the boundary of the area, *B*.

The inner rose, Area *A*, shows the number of hours, for the twelve years—1916 to 1927, inclusive—during which the wind blew at more than 56 miles per hour.

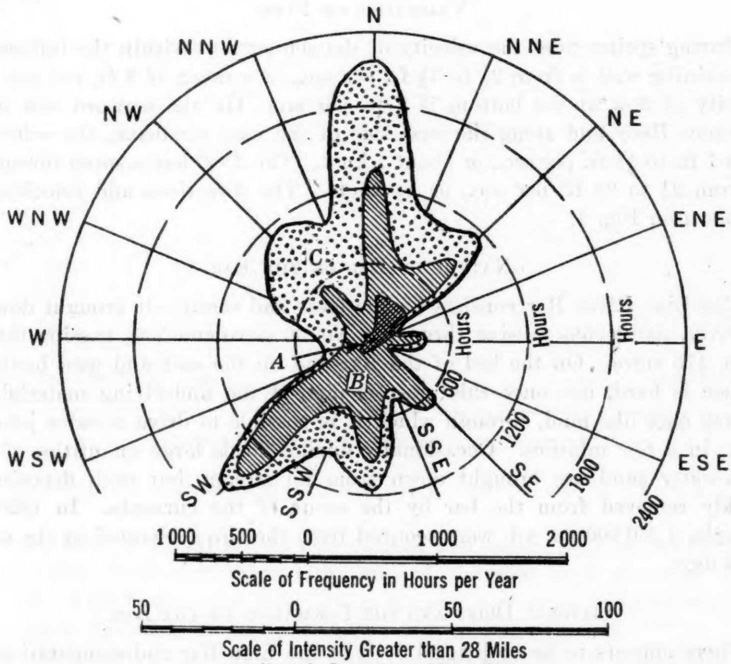

FIG. 6.—ANNUAL WIND FREQUENCY AND INTENSITY DIAGRAMS FOR YEARS 1916–1927, OBSERVED AT NIUCHUANG CUSTOMS.

A measure of wind intensity is given by the area, *C*, which does not include the areas, *A* and *B*. Measured radially, with the intensity scale, it indicates the percentage of the total wind frequency during which the wind measured from the area, *B*, blew with a velocity greater than 28 miles per hour.

The Port of Niuchuang is closed by ice from December to the end of March, during which period the wind is in the northeast quarter. The direction of storms during the period of open port is south to south 22° west and the prevailing wind southwest.

CURRENTS

The ebb and flood currents across the Liao River Bar are shown on Fig. 4. Since the jetty is exposed for only about two hours before low water, the first of the ebb crosses it in a direction, S. 30° to 35° W. (true meridian). After the jetty has been uncovered, the current becomes more parallel with it, that is, about S. 55° W.

On the seaward side of the entrance buoy and beyond the influence of the jetty, the ebb currents flow S. 16° to 33° W.; the more southerly direction being the ebb current flowing down and practically parallel with the west side of the west sandbank. The flood tides flow in a direction from N. 25° to 35° E.

Velocities of Flow

During spring tides the velocity of the ebb current within the influence of the training wall is from $2\frac{1}{2}$ to $3\frac{1}{2}$ ft. per sec., or a mean of 3 ft. per sec. The velocity of flow at the bottom is 2 ft. per sec. On the seaward side of the Entrance Buoy and along the west side of the west sandbank, the velocity is from 1 ft. to $1\frac{1}{2}$ ft. per sec., or about 1 knot. The flood has a speed throughout of from $2\frac{1}{4}$ to $2\frac{3}{4}$ ft. per sec., or $1\frac{1}{2}$ knots. The directions and velocities are as shown on Fig. 4.

Nature of Bed of the Bar

The Liao River Bar consists of fine loess and sandy silt brought down by the river, about 95% passing through a No. 30 sieve and 50% passing through a No. 175 sieve. On the bed of the Bar and on the east and west banks the surface is hard, but once this crest is pierced the underlying material is a gummy clay like mud, through which it is possible to drive a water jet down 20 ft. in a few minutes. Occasionally during floods large quantities of pure fresh-water sand are brought down from up stream, but such deposits are quickly removed from the bar by the scour of the currents. In 1920, for example, 1 250 000 cu. yd. were scoured from the ships' channel in the course of 15 days.

Littoral Drift and the Formation of the Bar

There appears to be no littoral drift at the Liao Bar and comparative surveys (see Fig. 5) do not show a definite relation between storms and changes in the low-water contours or depths. Again, contrary to the generally expressed opinion that bars at the mouths of tidal rivers are formed of the materials thrown up from the ocean bed during storms, those at the entrances of the Liao, Hai, and Yellow Rivers, flowing into the Gulf of Pechihli, are all formed by alluvia brought down by the rivers themselves and deposited in the sea in a similar manner to deltaic formations at the mouths of non-tidal rivers. The bars of the Liao and Hai Rivers both shoal from 1 ft. to 2 ft. during the freshet season which occurs in August and September, the period of calm weather. As far as can be ascertained storms appear to have very little effect upon the Liao Bar.

Improvement of the Bar by the Construction of the East Jetty Only

Fig. 7 shows three longitudinal profiles across the bar and along ships' courses as they were in the years 1917, 1927, and 1928 (see Fig. 5). In 1917, one year after construction had begun on the east wall, the bar was 2 miles

long. At that time the crossing channel was only 6 ft. deep and during the greater part of the year it was only 5 ft. deep at low water of ordinary spring tides.

When a second survey was made in 1922 the wall had been constructed to a point opposite the Middle Buoy. Originally, this was to have been the end of two training walls, which were to have been assisted by a dredge.

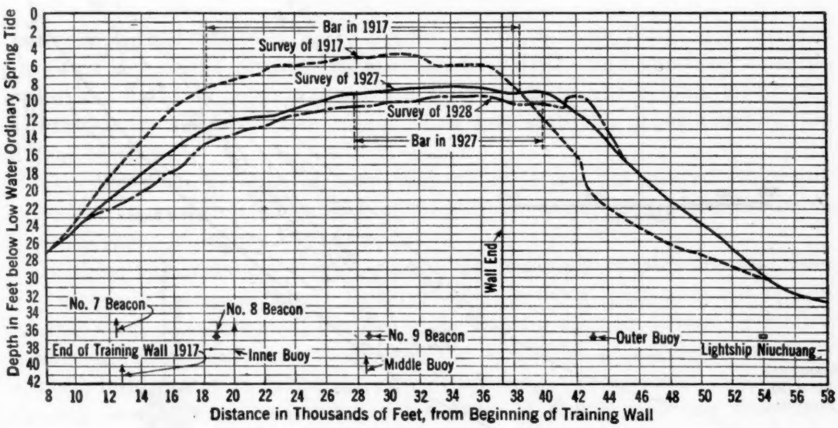

FIG. 7.—LONGITUDINAL SECTIONS ALONG LINE OF SHIPS' COURSE DURING 1917, 1927, AND 1928.

At that time, however, the old east channel had merely been pushed seaward and was still continuing around the end of the wall in a direction about S. 16° W. This is the same direction as the ebb currents in this vicinity and is in a mean direction of the prevailing winds and the storms.

The third survey made in 1924, after the wall had been extended still farther in a direction toward the Entrance Buoy, showed this easterly channel still persisting. During 1923, following a heavy freshet, the main body of the bar and ships' channel silted 2 ft. more than this easterly channel. Consequently, the easterly channel was made into the ships' course for a period of about one month until the silt and sand had been scoured off the bar. Then the old course was resumed and the construction of the wall was continued.

In 1926 the training wall was 6 miles long and a survey showed that the east channel was practically closed. An examination of these four surveys (1917, 1922, 1924, and 1926) showed that a depth of 9 ft. has always been obtained up to the existing end of the jetty and that by 1926 there was a navigable depth of 8 ft. across the bar. The positions of the bar in 1927 and 1928 are shown in Fig. 7. In 1928 the bar had a minimum navigable depth of 9 ft. and the wall was 37 750 ft. long.

The present plan is to terminate this wall in 14 ft. at low water opposite the Entrance Buoy (see Fig. 6). From Fig. 7 it will be noted that the bar has been advanced seaward into deeper water.

Advance of the 9-Foot Contour

Fig. 8 shows the 9-ft. contour for a number of years from 1917 to 1928. The total advance is 22 500 ft. and has always followed the advance of the training wall. In fact, about 1 000 ft. up stream from the position of the contour for any year can be assumed to be the end of the wall for that year.

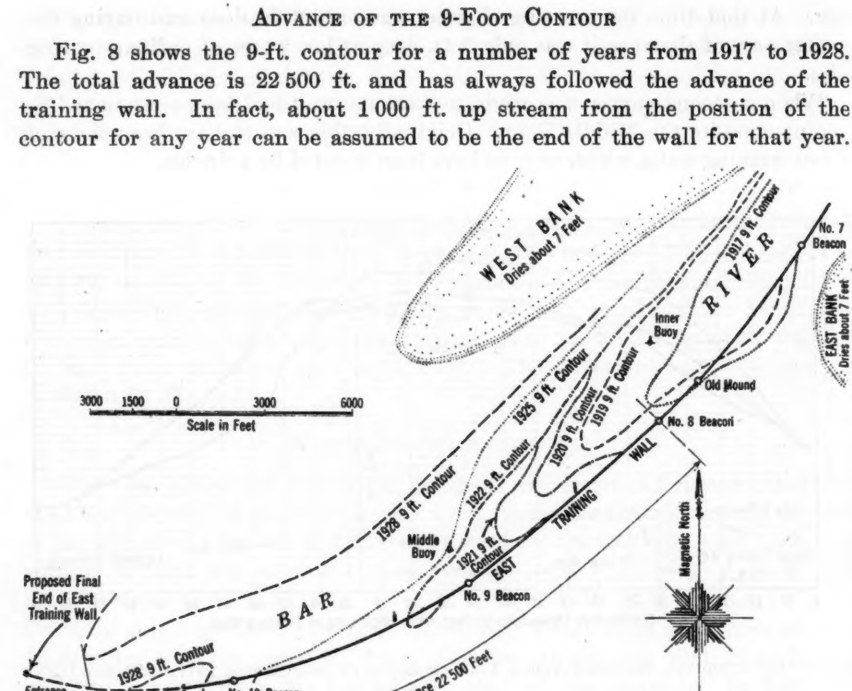

Fig. 8.—Liao Bar: 9-Foot Contour from 1917 to 1928.

The width from the wall to the west side of the contour (about 3 000 ft.) shows that the deepening has not been caused by over-fall during flood tide. The flood tide tops the wall after its first half hour of flow and passes over it very quietly and without any great disturbance. In another part of the river there is constructed a wall across what was known as the west channel. In this locality the early flood tide passes over with a head of about 1 ft. and causes a deepening on the river side due to over-fall. This deepening, however, is very narrow (about 100 ft.) and an excessive shoaling is formed in front of it.

Were the deepening across the bar caused by over-fall it would be expected to find a corresponding shoaling in the main channel. The contrary is the case; the deepening that is taking place in the vicinity of the west bank extension is a disquietening feature because it is considered that should the channel deepen over too great a width, a shoaling will take place and necessitate the construction of a west wall. The deepening across the bar has undoubtedly been caused by the scour of the ebb tide excavating the bar and depositing it seaward into deep water.

Quantity Scoured Off the Bar

The quantity scoured from the bar—irrespective of the annual freshet deposits—is equal to 22 000 000 cu. yd.

The annual freshet deposit is at times very great. In the freshet of 1923, surveys showed that a quantity of 1 250 000 cu. yd. of clean sand were removed in 15 days, or at the rate of 80 000 cu. yd. per day. Had such a deposit taken place prior to the construction of the wall, the bar would have shoaled for a year, or probably more, since there would have been no confined and trained ebb current to scour it away.

Height of the Wall

In the United States it would appear that high-water training jetties are advocated and almost invariably constructed. British engineers favor low-tide jetties, or, half-tide jetties, because a jetty built to above high water will interfere with the free entry of the flood tide into the upper parts of the estuary or river.

The American practice of twin high-water jetties has proved eminently successful in numerous instances, of which two examples are the Columbia River entrance and Coos Bay, Oregon.

In these two instances single jetties were originally constructed and were very successful for some time. Unfortunately, however, they were not carried out into deep enough water; that is, to the neutral zone as defined by Italian engineers. The result was that after the sand had accumulated on their windward side so as to fill in the space provided, it commenced to enter the channel, thus causing the latter to deteriorate. A second jetty was then built extending out into deep water and the original jetty was also lengthened.

This work met with great success, but it seems unfortunate that the original single jetties were not extended first, so as to pass the sand drift around their outer ends, and so that the effects could be observed. By this means equally successful results might have been achieved without having recourse to the second jetty.

In the case of the Liao entrance the east and west banks form what are really two low-tide jetties which maintain a channel between their 20-ft. contours over a width of almost 2 000 ft. It would appear reasonable to expect two low-tide walls to act in a similar manner if extended seaward.

Irrespective of this point of view and because of ice drifting across the jetty during the winter months, it has been found that the wall across the Liao Bar must be either practically a low-tide one, or that it must be built completely above high water.

The present method is achieving immediate results while at the same time it permits the jetty to be carried above high water, should that be deemed advisable at any future date when the trade of the port becomes sufficient to justify the expense. Model tests would be invaluable in solving several problems of this nature.

The solution of the problem became a question of attempting to get the best results for a minimum expenditure. Since the whole body of the ebb tide travels in a south-southwesterly direction, or away from the west jetty (if that is ever built), and since it was seen that the low tide east jetty always deepened the bar to 9 ft., as its construction advanced, it was decided to continue extending the east jetty and to hold the construction of a west jetty in abeyance.

Before arriving at this decision, the question of costs was analyzed. To date, the cost has been approximately \$1 250 000 (gold). Maintenance charges are likely to equal about \$60 000 (gold) per year, whereas the cost of a suitable drag suction dredge would be \$650 000 (gold) and operating expenses \$60 000 (gold) per year.

It would appear, therefore, to be more economical to abandon the intention of building the second jetty or heightening the present one and to rely on any further deepening to be obtained by dredging. This would save half the original first cost of the west jetty—or all the cost of heightening the east jetty—plus the additional interest and amortization charges. The latter would be sufficient to provide a new dredging unit every ten years.

Comparative Surveys

Comparative surveys previously mentioned, show that the 1-fathom depths have had an eastward advance of about 5 400 ft. between 1860 and 1927, or at the rate of about 80 ft. annually. The direction of the channel in 1860 and in 1889 was S. 53° W.; in 1907 it was southwest; and in 1927 the channel trained by the east jetty was S. 67° W. Since 1907 the old east channel has been on the east side of the jetty. It is now filling rapidly with silt.

The writer has also studied the relative movement of the 2, 3, 4 and 5-fathom contours. They are of interest inasmuch as they show that the advance of the silt takes place southeast of the Entrance Buoy. The 3-fathom contour has moved seaward a distance of 8 000 ft. in 67 years, or at the rate of 120 ft. annually.

Furthermore, during the whole of this period, the contours north of the Entrance Buoy on the west side of the west sand bank have maintained their position with very little change. This is believed to be due to the action of the flood and ebb tides passing straight up and down along these contours in a north and south direction, respectively, and thus carrying away any deposits that may be made from time to time in this locality.

Location of the Wall

Hydraulic engineers are not in agreement as to the necessary direction and alignment of training walls across bars subject to tidal action. Some engineers recommend that the end of the wall be in line with and along the axis of the flood and ebb currents and directly facing into the direction of the prevailing winds and storms.*

Others favor inclining the wall away from the prevailing winds and heaviest seas and locating it so that these and the flood tide meet the wall slightly on its convex side.† This is the location that has been adopted in the case of the Liao Bar.

The survey made in 1927 shows clearly the steep slopes of the sea bed north of the entrance buoy (see Fig. 9, Curve *BDF*). The other surveys since 1860 show these slopes to have been maintained at the same location without

* See Bibliography (Appendix), Items (3), p. 121; (6), pp. 303-304; (14), p. 354; (20), p. 194; (21), p. 99; (22); and (23), p. 207.

† See Bibliography (Appendix), Items (4), p. 485; (5), p. 536; (7), p. 6; (10), p. 10; (1); and (2).

practical alteration since that date. On the other hand, the sea bed slopes east of the jetty, along the axis of the east channel, and along the direction of the flood and ebb currents and the direction of prevailing winds and storms, are extremely flat, as shown by the longitudinal section, *C E F*, in Fig. 9.

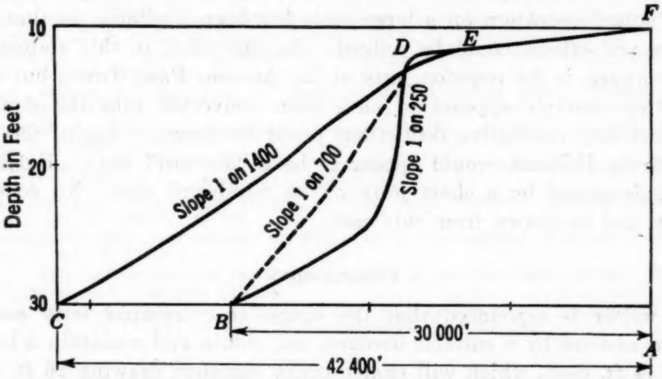

FIG. 9.—LONGITUDINAL SECTIONS NORTH OF THE ENTRANCE BUOY, LIAO BAR.

The question to be considered, therefore, was the correct direction to adopt. The old east channel course appears to be the natural direction of both the flood and ebb currents. This route would have provided a channel facing directly into the prevailing winds and greatest storms as advocated by some authorities; but it would also have led along a line of flat sea-bed slopes in a direction such that the greatest accretion and contour advance due to the deposits from the river would have been taking place.

The other alternative was to oppose the ebb currents by the training wall and attempt to guide the channel so as to terminate at or north of the Entrance Buoy in a locality of steep sea-bed slopes. As previously stated (see Fig. 9), the contours have here remained practically stationary for as far back as there are any records, that is, 1860. This procedure, on the other hand, would necessitate the flood tide, prevailing winds, and greatest storms being received on the convex side of the wall. Furthermore, the ebb currents on passing the end of the wall would have to make a more or less oblique bend to the southwest.

The pressure of the ice drift during the winter months and the abrasion against a wall that opposes the ebb currents in any way is very great and this no doubt increases the maintenance charges. Nevertheless, it was considered best to end the wall where the sea bed was steep because any other location would probably make it necessary to extend the wall continually to keep ahead of shoals formed by deposits brought down by the river. It was after consideration of these various factors that the present direction was adopted.

SINGLE OR DOUBLE TRAINING JETTIES

The question as to the advisability of constructing single or double training jetties at the entrances of tidal rivers for the purpose of providing a

navigable channel across bars has been frequently discussed before the Society. Many eminent engineers have advocated a single jetty* and others, equally eminent, have advocated double jetties.†

The great difficulty has been that apparently no example of a single jetty in actual operation on a large scale has been available, so that relative behaviors and effects could be judged. An exception to this statement, the writer is aware, is the reaction jetty at the Aransas Pass, Texas, but unfortunately, this example appears to have been converted into the double-jetty type before any conclusive deductions could be drawn. Again, the jetty at Swinemünde, Holland, would appear to be a "leeward" jetty. Furthermore, it is supplemented by a short jetty on its windward side. No conclusions, therefore, can be drawn from this case.

Conclusions

The writer is convinced that the single east training jetty across the Liao Bar, assisted by a suitable dredger, can obtain and maintain a low-water channel 14 ft. deep, which will enable ocean steamers drawing 26 ft. to enter and leave the port of Niuchuang. This is the extent of improvement required in this particular case. As it is, at present vessels with a draft of as much as 22 ft. enter and leave the port, whereas previous to the construction of the jetty, 17½ ft. was the limiting draft.

APPENDIX

Bibliography on Single and Double Training Walls

The following is a bibliography of references favorable to single and to double training walls.

(1) **Brooks, William A.** On the Improvement of Tidal Navigations and Drainages. *Minutes of Proceedings*, Inst. C. E., Vol. XII (1852–53), pp. 1-23.

(2) **Wheeler, W. H.** Tidal Rivers. Longmans, Green and Co., Lond. and N. Y., 1893.

(3) **Symons, Thomas W.** Improving the Entrance to a Bar Harbor by a Single Jetty. *Transactions*, Am. Soc. C. E., Vol. XXXVI (1896), pp. 109 and 121. With discussion, p. 131.

(4) **Haupt, Lewis M.** The Reaction Breakwater as Applied to the Improvement of Ocean Bars. *Transactions*, Am. Soc. C. E., Vol. XLII (1899), p. 493. With discussion, pp. 503, 512, 516, 520; also, Vol. XLIII (1900), p. 102.

(5) **Harts, William W.** Description of Coos Bay, Oregon, and the Improvement of Its Entrance by the Government. *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), p. 482. With discussion, pp. 526, 532.

(6) **Gillette, Cassius E.** Sea Coast Harbors in the United States. *Transactions*, Am. Soc. C. E., Vol. LIV, Pt. A (1905), pp. 317–321.

(7) **Vernon-Harcourt, L. F.** Construction and Maintenance of Seaports on Sandy Coasts. *Bulletin No. 61*, Inter. Nav. Congress (1908), p. 6.

(8) **Sanford, J. J.** Construction of Ports on Sandy Shores. *Bulletin No. 62*, Inter. Nav. Congress (1908), p. 8.

(9) **Gatto, D. Lo.** Construction of Ports on Sandy Shores. *Bulletin No. 63*, Inter. Nav. Congress (1908), p. 7.

(10) **Ivanina, A., and Aeckerle, W. A.** The Most Important Russian Ports on the Sandy Coasts of the Baltic. *Bulletin No. 65*, Inter. Nav. Congress (1908), p. 10.

(11) **Harts, William W.** Harbour Improvement on the Pacific Coast of the United States. *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXIII (1910–11, Pt. 1), pp. 237 and 239.

* See Bibliography (Appendix), Items (1); (2); (3), pp. 109, 121; (4), pp. 493, 512, 516, 520, 102; (12); (13); (14); and (23).

† *Loc. cit.*, Items (3), p. 131; (4), p. 503; (5) to (11); (15) to (20).

(12) **Halligan, G. H.** The Bar Harbors of New South Wales. *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXIV (1910–11, Pt. II), pp. 128 and 205.

(13) **Wheeler, W. H.** Correspondence on Harbours in Australia. *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXIV (1910–11, Pt. II), p. 277.

(14) **Tower, Morton L.** Notes on the Bar Harbors at the Entrances to Coos Bay, and Umpqua and Siuslaw Rivers, Oregon. *Transactions*, Am. Soc. C. E., Vol LXXI (1911), p. 357.

(15) **Bagnall, Gerald.** The Improvement of the Mouth of the Columbia River. *Professional Memoirs*, U. S. Corps of Engrs., Vol. 8 (1916), p. 718.

(16) **Hickson, R. E.** Changes at Mouth of Columbia River, 1903 to 1921. *Military Engineer*, Vol. 14 (1922), p. 211.

(17) **Black, William M.** Formation of Channels Across Ocean Bars. *Military Engineer*, Vol. 14 (1922), p. 375.

(18) **Patton, R. S.** Problems Involved in Coast Erosion. *Military Engineer*, Vol. XVI. No. 90 (1924), p. 463.

(19) **Kuentz, Oscar O.** Cape Fear River Channel and Bar. *Military Engineer*, Vol. XVII, No. 91 (1925), p. 29. With discussion, p. 30.

(20) **Mayo, George.** The Improvements at Coos Bay, Oregon. *Military Engineer*, Vol. XVII, No. 93 (1925), p. 194.

(21) **Townsend, C. McD.** Problems at Mouth of the Mississippi. *Military Engineer*, Vol. XVIII, No. 98 (1926), p. 99.

(22) —— The Improvement of the Southwest Pass of the Mississippi River. *Bulletin No. 33*, Inter. Nav. Congress (1926).

(23) **Ripley, Henry C.** Relation of Depth to Curvature of Channels. *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 235. With discussion, pp. 244–245.

[illegible]

(5) Turner, Charles L. [illegible] Vol. LXXI (1957).

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PAPERS AND DISCUSSIONS

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EVAPORATION AS A FUNCTION OF INSOLATION

By Burt Richardson,* Esq.

Synopsis

In the Western States water is scarce and valuable, and rates of evaporation are high. Heretofore, evaporation losses have been ascertained by direct measurements. The writer considers it possible to approach the problem by a study of the physical cause of evaporation. The principal cause is insolation which may be defined, briefly as "exposure to the rays of the sun."

This paper is concerned with the effect of insolation and with the measurement of the quantity of radiant heat imparted by the sun and sky to the water and the bottom of a reservoir. In a sense, this effect may be considered a measure of insolation. For example, it may be said that, in Pasadena, Calif., on a sunny day, the "insolation" is roughly 600 calories per sq. cm. per day.

Introduction

The quantitative effect of insolation is variable and depends on the sun's elevation, the length of the day, cloudiness, smoke, etc. It may be determined by three independent methods:

(1) By means of a Weather Bureau, recording, thermo-electric pyrheliometer.

(2) By computing the quantity as a geometric problem from a knowledge of the sun's radiation, altitude, etc., together with the latitude of the reservoir or lake.

(3) By tracing the radiant heat energy which strikes the water surface of a heat-insulated tank.

Having determined the value of insolation for a particular site it is possible to show what part of it goes into evaporation and to place definite limits on the

Note.—The Special Committee on Irrigation Hydraulics selected the subject of "Evaporation Losses from Reservoirs" as one of ten for study and research. The author was urged to prepare a paper on this subject. After its submission the Committee recommended its publication in *Proceedings* in order to elicit discussion on this new method of determining evaporation losses from water surfaces. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 176). Written discussion on this subject will be closed in September, 1930, *Proceedings*.

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quantity of evaporation. By using either Method (1) or Method (2) in combination with Method (3) the evaporation from a lake or proposed reservoir may be determined. Attention is called to the fact that there is a regrettable scarcity of reliable data for evaporation studies. The data desirable for such studies include:

(*a*) Daily, monthly, and yearly records of evaporation from lakes and pans all observed in a uniform manner.

(*b*) Insolation records, especially in country districts and in the Western States (the present six U. S. Weather Bureau Stations at which insolation records are secured are located in cities).

(*c*) Records of the temperature gradient of lakes.

(*d*) Cloudiness records kept in a uniform manner (at present, there is no unit for cloudiness, and it is impossible to convey to another the meaning of a cloudy or very cloudy day).

(*e*) Records of transmission coefficients of the atmosphere for various sections of the country to ascertain if there exists a "mean weighted transmission coefficient" for a certain district.

(*f*) Data concerning water surface and air temperatures of lakes to provide a means of computing radiation, which is a factor in the evaporation equation.

Notation

The notation adopted for use in this paper is as follows:

A = correction coefficient = $\dfrac{\text{solar radiation}}{\text{total radiation}}$.

a = the radiation constant introduced by Stefan-Boltzmann.

B_r = the radiation from water.

b = a constant which permits the use of the air temperature above a body of water instead of the sky temperature to which the radiation actually goes.

C = a factor representing the conduction of heat through the walls of the tank.

C' = the percentage of cloudiness that occurred during a certain month, as given in Monthly Weather Review reports.

d = the sun's declination for a certain day of the year.

E = quantity of water evaporated.

F = factor denoting the relative clearness of the sky above the particular body of water in question.

H = original vapor pressure of the air passing over the surface of the water.

i = angle of incidence.

I = the quantity of radiant heat, in calories per minute, that reaches a certain area on the surface of the earth.

I_{ex} = insolation incident upon the exterior of the earth's atmosphere.

K = the solar constant.

k = ratio of the incident to the reflected intensity of radiation.

L = latent heat of evaporation.

n = index of refraction.

P = atmospheric pressure.

P_w = original vapor pressure of the air contact with the water surface.

Q = the observed radiant energy that passes through local clouds.

Q_s = the observed radiant energy that passes through a clear sky.

R = the ratio, convection evaporation.

r = angle of refraction.

S = the sensible heat measured by the warming or cooling of the water.

T_a = original temperature, centigrade, of the air passing over the surface of the water.

T_w = original temperature, centigrade, of the air in contact with the water surface.

t = time, in minutes.

$w = \dfrac{2\pi}{24 \times 60}$ radians per min. = angular velocity of the earth on its axis.

λ = the latitude of the station.

ρ = the transmission coefficient of the earth's atmosphere.

ρ' = the mean transmission coefficient of the atmosphere at a particular place.

ψ = the zenith angle of the sun.

(1) Insolation Recorded by Thermo-Electric Pyrheliometers

The Weather Bureau of the U. S. Department of Agriculture maintains six stations at which recording thermo-electric pyrheliometers* are used and their records of insolation are published in the *Monthly Weather Review*. Due to the difference in latitude, cloudiness, etc., the value of insolation is, of course, not the same for all stations. For example, the average number of calories reaching each square centimeter of the earth's surface per day during the month of July, 1928, at certain places is indicated in Table 1.

TABLE 1.—Values of Insolation at Seven Stations in the United States.

Location.	Insolation, in calories per square centimeter per day.	Location.	Insolation, in calories per square centimeter per day.
New York, N. Y.	378.5	Lincoln Nebr.	531.2
Washington, D. C.	540.8	Twin Falls, Idaho	737.2
Chicago, Ill.	412.8	Scripps Institute of Oceanography, La Jolla, Calif.	544.7
Madison, Wis.	509.2		

The insolation varies also from day to day at any one station, as illustrated by data observed at the Scripps Institute of Oceanography at La Jolla, Calif. While this is not a U. S. Weather Bureau Station, it uses the U. S. Weather Bureau Pyrheliometer, No. 17. Observations for each day of July, 1928, are given in Table 2.

The total insolation for the 31 days was 16 886.3 calories, which is the equivalent of 10.35 in. of evaporation, or an average of 544.7 calories per day.

Considering that it requires 585.4 calories to evaporate 1 cu. cm. of water at 20° cent., it is easily seen how much a given water surface would drop each day due to evaporation, should all the insolation go into evaporation. Actually, the ratio of observed evaporation to observed insolation is between 40 and 60 per cent.

Table 3 gives all the records of insolation that have been observed for stations in the United States. It has been compiled from various sources. Items

* H. H. Kimball and H. E. Hobbs, *Journal*, Optical Soc. of America, Vol. 7, September, 1923.

Nos. 1 to 18, inclusive, are records at six stations of the U. S. Weather Bureau where insolation is observed;* Items Nos. 19 to 42, inclusive, are records reported from sundry other stations;† and the observations listed for the last five stations are those observed by the writer, using the pan method.‡

TABLE 2.—INSOLATION AS RECORDED BY THE U. S. WEATHER BUREAU PYRHELIOMETER NO. 17, AT SCRIPPS INSTITUTE OF OCEANOGRAPHY, LA JOLLA, CALIF.

Date, July, 1928.	Insolation, in calories per square centimeter per day.	Date, July, 1928.	Insolation, in calories per square centimeter per day.
1	662.2	16	600.6
2	627.1	17	604.1
3	669.1	18	579.6
4	679.1	19	632.3
5	620.6	20	481.9
6	614.0	21	534.2
7	610.3	22	432.8
8	591.3	23	353.6
9	665.9	24	275.5
10	605.1	25	238.0
11	586.5	26	517.4
12	471.9	27	592.4
13	482.9	28	489.2
14	633.6	29	474.7
15	571.4	30	410.0
		31	579.0

(2) INSOLATION COMPUTED AS A GEOMETRIC PROBLEM

It is possible to determine the value of insolation from a knowledge of the sun's radiation, its altitude, etc., and the latitude of the reservoir or lake. A formula may be set up for insolation, as follows:

$$I = K \int_{t_1}^{t_2} \cos \psi \, \rho^{\sec \psi} \, d t \dots\dots\dots\dots\dots (1)$$

Differentiating with respect to time to obtain the rate of insolation,

$$\frac{d I}{d t} = K \cos \psi \, \rho^{\sec \psi} \dots\dots\dots\dots\dots (2)$$

The solar constant, K, as determined by Abbot,§ is 1.93 calories per sq. cm. per min. To illustrate the use of Equation (2) consider the parallel of 40° N. latitude. Use Abbot's value for the solar constant, ($K = 1.93$); for simplicity assume a transmission coefficient of $\rho = 0.7$, and insert proper values of ψ for each minute of each day of each month, successively. The values of I thus obtained are given in Table 4.

At this point something must be said about ρ, the transmission coefficient§ of the earth's atmosphere used in Equation (2). The only available values of this coefficient are as shown in Table 5. These were weighted values com-

* "Average of Values for a Particular Month Printed in *Monthly Weather Review* from Date of First Observation at a Station until January, 1929," *Monthly Weather Review*, Vol. 57, January, 1929.

† *Monthly Weather Review*, October, 1928, p. 430.

‡ National Research Council, *Bulletin No. 68*, April, 1929, and *Monthly Weather Review*, July, 1929, p. 300.

§ *Annals*, Astrophysical Observatory of the Smithsonian Institution, C. G. Abbot, Vol. 4, 1922.

TABLE 3.—THE MEAN MONTHLY QUANTITIES OF OBSERVED INSOLATION. ALSO, THE MEAN MONTHLY INSOLATION COMPUTED BY EQUATION (12) INCIDENT UPON THE EXTERIOR OF THE EARTH'S ATMOSPHERE. THE RATIO OF THE OBSERVED INSOLATION TO THAT COMPUTED ON EXTERIOR OF EARTH'S ATMOSPHERE.

Item No.	Station.	Position.		Nature of data.	Insolation, in Calories per Square Centimeter per Day.											
		North latitude.	West longitude.		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
1				Observed	128	235	262	354	874	358	402	276	264	195	114	91
2	New York, N. Y.	40°–46′	....	Computed	373	560	650	793	964	1 000	1 012	888	700	518	377	297
3				Ratio	0.343	0.42	0.404	0.446	0.388	0.358	0.397	0.311	0.377	0.377	0.303	0.306
4				Observed	178	242	338	419	472	489	473	420	360	279	188	143
5	Washington, D. C.	38°–56′	....	Computed	377	577	678	820	994	1 020	1 022	894	705	543	410	326
6				Ratio	0.472	0.42	0.50	0.51	0.475	0.483	0.468	0.47	0.51	0.51	0.46	.44
7				Observed	98	151	223	328	432	335	390	380	336	167	87	78
8	Chicago, Ill.	41°–47′	....	Computed	317	547	643	785	961	1 019	1 036	890	683	510	363	286
9				Ratio	0.31	0.278	0.347	0.418	0.45	0.320	0.375	0.427	0.493	0.328	0.24	0.273
10				Observed	165	245	315	414	450	519	505	434	334	221	139	125
11	Madison, Wis.	43°–05′	....	Computed	303	534	629	777	949	1 003	1 085	891	684	524	357	279
12				Ratio	0.545	0.46	0.50	0.532	0.475	0.519	0.465	0.487	0.49	0.421	0.39	0.45
13				Observed	208	300	374	451	475	556	570	486	392	290	203	171
14	Lincoln, Nebr.	40°–50′	....	Computed	324	561	650	794	965	999	1 020	889	690	515	379	298
15				Ratio	0.643	0.534	0.575	0.568	0.492	0.556	0.564	0.548	0.568	0.563	0.536	0.573
16				Observed	162	261	250	345	380	614	717	708	567	390	180	150
17	Twin Falls, Idaho	42°–27′	....	Computed	308	550	647	787	954	1 018	1 024	887	688	504	354	280
18				Ratio	0.526	0.475	0.387	0.44	0.40	0.60	0.70	0.798	0.823	0.774	0.51	0.536
19				Observed	81	147	204	283	366	357	326	304	199	181	74	58
20	Vancouver, B. C., Canada	52°–0′	129°–0′	Computed	196	342	549	792	943	1 006	942	791	571	347	196	153
21				Ratio	0.41	0.43	0.37	0.36	0.39	0.35	0.35	0.38	0.35	0.38	0.38	0.35
22				Observed	94	157	259	339	414	405	388	335	255	153	90	71
23	Near Victoria, B. C., Canada	48°–0′	124°0′	Computed	249	396	592	815	957	1.011	954	818	618	403	253	202
24				Ratio	0.38	0.40	0.44	0.42	0.43	0.40	0.41	0.41	0.41	0.38	0.36	0.35
25				Observed	114	206	270	345	397	424	422	364	284	165	108	78
26	Near St. Johns, N. B., Canada	48°–0′	60°–0′	Computed	249	396	592	815	957	1 011	954	818	618	403	254	202
27				Ratio	0.46	0.52	0.46	0.42	0.42	0.42	0.44	0.45	0.46	0.41	0.43	0.39

TABLE 3.—(*Continued*).

Item No.	Station.	Position. North latitude.	Position. West longitude.	Nature of data.	Insolation, in Calories per Square Centimeter per Day. January.	February.	March.	April.	May	June.	July.	August.	September.	October.	November.	December.
28				Observed	139	228	327	402	449	477	421	376	317	235	153	120
29	Boat off Boston, Mass.	42°-0′	66°-70′	Computed	331	477	655	848	964	1 014	965	850	673	475	332	282
30				Ratio	0.42	0.47	0.50	0.47	0.47	0.47	0.44	0.44	0.47	0.50	0.46	0.43
31				Observed	148	246	313	435	486	524	442	382	318	238	151	129
32	Near Eureka, Calif.	42°-0′	124°-0′	Computed	331	477	655	848	964	1 014	965	850	673	475	332	282
33				Ratio	0.45	0.52	0.48	0.51	0.51	0.52	0.46	0.45	0.47	0.49	0.46	0.46
34				Observed	306	364	415	482	476	482	449	437	420	340	301	281
35	Lower California, Mexico	30°-0′	117°-0′	Computed	494	615	761	898	973	997	978	898	774	618	496	453
36				Ratio	0.62	0.59	0.55	0.54	0.49	0.48	0.46	0.49	0.54	0.55	0.61	0.62
37				Observed	212	247	365	420	462	441	432	399	326	259	244	197
38	Boat off Florida	30°-0′	65°-77′	Computed	494	615	761	898	973	997	978	898	774	618	496	458
39				Ratio	0.43	0.40	0.48	0.47	0.47	0.44	0.44	0.44	0.42	0.42	0.49	0.43
40				Observed	335	420	446	475	452	493	464	450	428	379	324	307
41	Santiago, Cuba	20°-0′	77°-0′	Computed	624	728	832	922	950	957	950	920	840	726	621	589
42				Ratio	0.54	0.58	0.54	0.52	0.48	0.52	0.49	0.49	0.51	0.52	0.52	0.52
43				Observed	302	383	427	532	568	512	582	568	490	400	365	316
44	Pasadena, Calif	34.13°	....	Computed	430	545	704	849	960	1 011	986	902	779	616	474	418
45				Ratio	0.70	0.70	0.61	0.63	0.59	0.51	0.59	0.63	0.63	0.65	0.76	0.75
46				Observed	......	......	......	......	......	......	......	......	370	......	......	......
47	Fort Collins, Colo.	40.58°	....	Computed	......	......	......	......	......	......	......	......	647	......	......	......
48				Ratio	......	......	......	......	......	......	......	......	0.57	......	......	......
49				Observed	......	......	......	......	......	......	......	564	......	......	......	......
50	San Francisco, Calif	37.46°	....	Computed	......	......	......	......	......	......	......	874	......	......	......	......
51				Ratio	......	......	......	......	......	......	......	0.645	......	......	......	......
52				Observed	......	......	......	......	......	......	662	......	......	......	......	......
53	San Diego, Calif	32.75°	....	Computed	......	......	......	......	......	......	955	......	......	......	......	......
54				Ratio	......	......	......	......	......	......	0.70	......	......	......	......	......
55				Observed	264	316	365	433	487	504	545	438	285	291	298	275
56	La Jolla, Calif	32.79°		Computed	......	......	......	......	......	......	967	......	......	......	......	......
57				Ratio	......	......	......	......	......	......	0.564	......	......	......	......	......

puted from the mean solar intensity and coefficient for a particular wave length.

Values of the transmission coefficient for other places in the United States have not been computed so that research on this coefficient is highly desirable. The writer suggests that this coefficient may be computed directly by means of Equation (2). The left side of this equation which represents the rate of insolation is known from the pyrheliometer record by direct reading for a particular minute of the day, and the zenith angle* corresponding to this minute is also known, which leaves the one unknown, ρ.

TABLE 4.—COMPUTED VALUES OF I AT 40° NORTH LATITUDE.

Month.	Values of I, in calories per square centimeter per day.	Month.	Values of I, in calories per square centimeter per day.
January	140.0	July	553.3
February	244.2	August	457.3
March	356.5	September	347.7
April	465.7	October	240.0
May	573.7	November	136.0
June	586.7	December	100.1

Total energy, including scattered sky radiation, is recorded by a pyrheliometer. Therefore, it is necessary to correct the pyrheliometer records by a coefficient, A, which represents the ratio of solar radiation to total radiation. Typical values of A for four stations† are given in Table 5.

TABLE 5.—OBSERVED VALUES OF ρ AND A.

Station.	North latitude.	Values of ρ.	Values of A for different Zenith Angles, ψ.						
			0°	7.5°	25°	30°	48.3°	55°	60°
Mt. Wilson, Calif.	34°-13′	0.841	87	86	86	86	84	80	80
Washington, D. C.	38°-56′	0.689	87	85	84	84	83	81	80
Mt. Whitney, Calif.	....	0.882							
Madison, Wis.	40°-51′	....	87	86	85	85	84	82	81
Lincoln, Nebr.	43°-05′	....	87	86	84	84	84	79	76

The rate of insolation from the sun (not including sky radiation) equals:

$$\frac{d\,I'}{d\,t} = A\,\frac{d\,I}{d\,t} = K \cos\psi\;\rho^{\sec\psi} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and, making the necessary transpositions:

$$\rho = \log^{-1}\left[\frac{\log\left(\dfrac{A\,\dfrac{d\,I}{d\,t}}{K\cos\psi}\right)}{\sec\psi}\right] \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

* "American Practical Navigator," Bowditch.

† *Monthly Weather Review,* April, 1927, p. 156.

To illustrate this method for computing the transmission coefficient, the pyrheliometer records of fourteen different days of 1926 for Pasadena, Calif., were used, and when monthly values were plotted, a mean value for ρ of 0.751 was obtained.

The value of insolation is not constant along any parallel of latitude, but changes at points of different longitude, due to variation in the transmission coefficient, humidity, local clouds, smoke, dust, etc. Therefore, in order to use Equation (2) for points of different longitude, values of the transmission coefficient must be observed by Abbot's method or computed by Equation (4).

(3) Insolation by Tracing Radiant Heat Energy

The third method* for measuring insolation is to trace the heat energy which comes to water contained in an open tank. In this case insolation (when corrected for reflection) will be equal to the summation of: (*a*) Evaporation; (*b*) convection losses, due to the circulation of air above water; (*c*) sensible heat, measured by the warming of the water; (*d*) conduction, due to loss of heat through the walls of the tank; and (*e*) radiation, from the water to colder air. This is simply a statement of the Law of Conservation of Energy.

The ratio, k, of the incident to the reflected intensity of radiation is given by Fresnel's equation:

$$k = \frac{1}{2}\,\frac{\sin^2(i-r)}{\sin^2(i-r)} - \frac{1}{2}\,\frac{\tan^2(i-r)}{\tan^2(i-r)} \ldots\ldots\ldots\ldots\ldots (5)$$

in which, i and r are the angles of incidence and of refraction, respectively, and are connected by the relation, $n = \frac{\sin i}{\sin r}$ = index of refraction.

Let n, for water, be equal to 1.33; then r may be found for corresponding values of i and these, substituted in Equation (5), will give values of k for any angle of incidence. This is the correction factor for reflection.

Insolation is a positive quantity during the day, but at night it is equal to zero. Evaporation is always positive, while the other terms—convection, sensible heat, conduction, and radiation—may be either positive or negative. In symbols, then:

$$I = LE + LER + S + C + B_r \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which, L is the latent heat of evaporation (585.4 at 20° cent.); E is cubic centimeters of water evaporated; and R is the ratio, $\frac{\text{convection}}{\text{evaporation}}$.

Bowen† has found that:

$$R = \frac{0.46\,(T_w - T_a)\,P}{(P_w - H)\,760} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which, T_a and H are the original temperature and vapor pressure of the air passing over the surface of the water; T_w and P_w are the corresponding quantities for the layer of air in contact with the water surface; P is the

* N. W. Cummings and Burt Richardson, *Physical Review*, October, 1927.

† *Physical Review*, June, 1926.

atmospheric pressure; S is the sensible heat measured by the warming or cooling of the water; and C represents the conduction of heat through the walls of the tank.

B_r is the radiation from water* and may be computed by the formula:

$$B_r = (0.906\ a\ T_w^4 - b\ a\ T_a^4) \qquad (8)$$

in which, the decimal, 0.906, represents an important constant which takes care of the fact that water does not radiate as a perfectly black body; a is the Stefan-Boltzmann radiation constant, amounting to 5.7×10^{-5} ergs per sq. cm. per sec. per degree to the fourth power; while b permits the use of air temperature in the formula in place of temperature of the sky which receives the radiant heat energy. The average value of the constant, b, in the radiation Equation (8) has been observed at four stations, to be as follows:

Station.	Mean value of b.
Tank at Pasadena, Calif.	0.815
Tank at Fort Collins, Colo.	0.760
Murray Lake, San Diego, Calif.	0.794
Crystal Springs Lake, San Francisco, Calif.	0.757
Average (weighted)	0.790

When the temperature of the water and air is known, by using the mean value of 0.790 for b, radiation may be computed by Equation (8). However, when these temperatures were not known, it was possible, in the case of California water surfaces, to estimate the value of radiation from the data collected at other places. The radiation from the water to the sky, in calories per square centimeter per day, has thus been found for five places as shown in Table 6.

TABLE 6.—RADIATION FROM THE WATER TO THE SKY, IN CALORIES PER SQUARE CENTIMETER PER DAY.

Location.	Radiation.
Pasadena, Calif.	117.0
Fort Collins, Colo.	132.0
Murray Lake, San Diego, Calif.	128.0
Crystal Lake, San Francisco, Calif.	148.0
La Jolla, Calif.	130.8
Average value (weighted)	131.4

From Equation (6) the exact amount of radiation may be determined for the night, since insolation during the night is zero and all other quantities of the equation are measurable. Thus, the insolation may be determined by adding all the heat energy coming to the body of water. Table 7 demonstrates the order of magnitude of each item that enters into insolation. A similar tabu-

* National Research Council, *Bulletin No. 68*, April, 1929.

lation for observations at Murray Lake, San Diego, Calif. (July 7 to July 27, 1927), gave corresponding average daily values, as follows:

Factors.	Calories per square centimeter per day.
Evaporation	485.9
Sensible heat	0.8
Convection	34.4
Back radiation	128.2
Conduction	13.0
Observed insolation	662.3

TABLE 7.—ITEMS THAT MAKE UP PAN INSOLATION, AS OBSERVED AT CRYSTAL SPRINGS LAKE, SAN FRANCISCO, CALIF., AUGUST 10-30, 1927.
(In calories per square centimeter per day.)

Day.	Evaporation.	Sensible heat.	Convection.	Back radiation.	Conduction.	Pan observed insolation.
1	389.1	0	103.2	147.1	19.3	658.7
2	232.5	— 1.9	40.5	145.5	17.6	434.2
3	438.9	— 28.0	51.1	141.3	12.5	615.8
4	403.0	13.0	6.7	144.5	9.6	576.8
5	351.8	83.7	26.0	148.0	14.4	623.9
6	494.2	09.4	101.4	171.4	30.6	707.0
7	355.5	— 9.3	94.5	165.0	27.6	633.3
8	326.4	— 3.6	87.7	162.5	26.2	599.2
9	395.3	33.5	66.6	158.8	20.2	674.4
10	412.2	— 11.2	14.3	139.0	10.5	564.8
11	393.4	0	— 8.0	144.0	2.8	532.2
12	392.5	063.2	19.2	136.2	10.9	622.0
13	327.4	48.4	10.5	139.0	11.1	536.4
14	311.5	— 5.5	22.6	146.5	13.1	488.2
15	381.8	— 13.0	15.3	145.8	13.1	543.0
16	370.2	— 9.4	29.8	141.5	11.2	543.3
17	315.0	7.5	65.4	150.2	15.1	553.2
18	243.8	— 40.8	33.6	136.1	8.7	381.4
19	383.4	63.3	19.5	139.6	8.2	614.0
20	379.2	26.1	24.6	142.5	12.1	584.5
Total, 20 days.....	7 197.1	225.4	824.5	2 944.5	294.8	11 486.3
Average per day..	359.9	11.3	41.2	147.2	14.7	574.3

As a test of this method, tanks of different areas and depths have been measured hourly over long periods of time with a resulting difference of less than 3 per cent. As one illustration, Table 8 will show a comparison of the results obtained from three tanks of different areas and depths and a Weather Bureau pyrheliometer (No. 17) over a period of five days, at the Scripps Institute of Oceanography, La Jolla, Calif.

TABLE 8.—COMPARISON OF INSOLATION QUANTITIES OBSERVED BY DIFFERENT METHODS.

Method of observation.	INSOLATION, IN CALORIES PER SQUARE CENTIMETER PER DAY.					Average.
	July 9.	July 10.	July 11.	July 12.	July 13.	
Pyrheliometer.....	662.5	614.2	583.0	468.2	476.2	560.8
Tank No. 1........	653.6	617.7	594.0	463.4	496.3	565.0
Tank No. 2........	629.0	590.3	565.2	445.9	480.7	542.2
Tank No. 3........	688.3	626.7	578.9	474.5	443.4	552.3

Description of Method for Measuring Evaporation

By transposing Equation (6) which is in metric units, and converting to inches, it may be written:

$$E = \frac{I - S - C - B_r}{2.54\ L\ (1 + R)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

This expression, which gives evaporation as a function of insolation, water temperature, wet and dry bulb temperatures, and vapor and atmospheric pressures, is a true equation; but the tedious process of evaluating each term hourly (as was done in the case of data mentioned in this paper) would defeat the purpose of this method for practical use. As a result of research, it is possible to assign limiting values to each term of Equation (9) and by considering certain variables constant, an equation may be obtained which is simple to handle. Thus, by considering sensible heat and conduction negligible, using 585 for the latent heat of evaporation, and 0.22 for Bowen's ratio, R, Equation (9) becomes, in inch units:

$$E = \frac{I - B_r}{2.54 \times 585 \times 1.22} = \frac{I - B_r}{1\ 814} \quad \ldots\ldots\ldots\ldots\ldots (10)$$

Thus far, no mention has been made of the effect of wind upon evaporation, because in the expression for insolation given by Equation (6) this effect would be measured, calorie for calorie, in changes in sensible heat and in convection. Sensible heat as well as the value of conduction for a short period or over a yearly cycle, as determined from a study of data collected in California, is negligible. Furthermore, no serious error will result from considering R as a constant equal to 0.22. This may be seen by referring to Table 9 which gives the values of R computed under dissimilar circumstances and over a long period of time with hourly readings.

TABLE 9.—Comparison of Values of R Computed Under Dissimilar Circumstances.

Location.	Date.	Interval, in days.	Ratio, R.
Pasadena, Calif.	June, 1926	10	0.193
Fort Collins, Colo.	September, 1926	10	0.221
Murray Lake, San Diego, Calif.	July, 1927	20	0.165
Crystal Springs Lake, San Francisco, Calif.	August, 1927	20	0.280
Scripps Institution Pier, La Jolla, Calif.	July, 1928	5	0.180
Pacific Ocean, Santa Babara Channel, Calif.	July, 1928	12	0.280
U. S. Naval Air Station: San Diego Bay, Calif.	August. 1928	31	0.190
San Diego Bay, Calif.	September, 1928	30	0.208
San Diego Bay, Calif.	November, 1928	30	0.171
San Diego Bay, Calif.	March, 1929	31	0.170
San Diego Bay, Calif.	April, 1929	30	0.152

The maximum deviation in these values of R from the mean is 0.06, and since evaporation equals 50% of insolation on the average, the resulting error in insolation caused by taking R equal to a constant, 0.22, instead of the largest or smallest value of R computed, would be 3 per cent.

TABLE 10.—OBSERVED EVAPORATION AT THIRTY-TWO STATIONS IN THE UNITED STATES, IN INCHES PER MONTH.

Station.	Relative location, in degrees north latitude.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total for one year.
Boston, Mass.*	42.5	0.9	1.20	1.80	3.10	4.61	5.86	6.78	5.49	4.09	2.95	1.63	1.20	39.61
Rochester, N. Y.*	43.0	0.86	0.86	1.67	2.39	3.45	4.38	4.80	4.54	3.54	2.60	1.46	1.21	31.76
Cincinnati, Ohio*	39.0	1.00	1.50	2.50	4.12	5.07	6.21	7.20	7.26	5.68	3.00	1.50	1.00	45.99
Birmingham, Ala.*	33.6	1.50	1.50	2.25	4.45	5.91	7.28	7.36	7.34	6.00	4.00	2.25	1.50	51.34
Great Lakes*	....	.66	0.84	1.03	2.12	2.76	3.67	4.38	3.98	3.21	2.56	1.67	1.08	27.96
Mitchell, Nebr.†	41.0	1.75	1.75	3.00	4.50	6.25	8.05	10.95	9.39	7.44	5.59	4.00	3.00	65.67
Snake River, Idaho†	42.5	2.25	2.50	4.00	7.00	11.21	12.31	15.00	13.50	11.00	8.50	5.75	3.50	96.52
Boise, Idaho†	43.6	2.00	2.75	4.25	6.00	7.90	9.59	10.59	12.16	9.25	5.42	5.52	2.00	77.43
Fallon, Nev.†	39.0	1.75	1.75	2.25	3.25	5.25	7.86	9.86	8.70	5.13	3.35	2.50	2.00	53.65
Hermiston, Ore.†	45.8	1.25	1.25	3.00	7.28	7.89	9.54	12.04	11.07	7.35	3.88	2.00	1.50	68.05
Klamath, Ore.†	42.2	0.50	1.25	3.57	6.64	7.15	6.99	8.01	9.21	6.13	2.50	1.00	0.50	53.45
North Yakima, Wash.†	46.8	1.75	2.50	6.25	7.91	8.36	8.90	10.74	9.41	5.51	3.15	2.00	1.50	67.88
Tahoe Lake, Calif.†	39.2	1.75	1.75	1.75	2.00	3.00	4.25	6.19	7.08	6.22	3.60	2.62	2.00	42.21
Salton Sea, Calif.†	33.4	3.41	5.09	5.95	8.75	10.50	13.00	14.03	12.19	12.08	9.24	5.96	5.25	105.45
Indio, Calif.†	33.4	3.18	5.08	7.50	12.05	15.84	16.11	16.34	13.78	12.87	8.91	5.17	3.00	119.83
Mecca, Calif.†	33.4	2.92	5.00	8.07	10.87	12.72	14.23	15.21	13.22	10.29	8.17	4.13	2.98	107.81
Brawley, Calif.†	33.0	3.05	5.00	8.00	10.74	13.79	13.68	14.14	11.26	10.15	6.99	4.09	2.66	103.55
Mammoth, Calif.†	33.0	4.24	5.67	8.99	12.02	15.52	16.75	18.00	13.73	12.16	9.49	5.26	3.70	125.53
Phoenix, Ariz.†	33.6	4.25	4.40	5.25	7.00	9.50	12.00	12.75	12.50	11.00	8.31	6.56	4.22	97.74
Lee's Ferry, Ariz.‡	....	1.74	3.52	5.87	7.16	11.70	13.70	13.70	11.33	8.85	6.29	3.98	1.91	89.75
Roosevelt Dam, Ariz.‡	....	2.29	3.10	5.33	7.32	10.37	12.67	12.84	10.56	8.60	5.78	3.52	2.13	84.02
Mesa Experiment Station.‡	....	2.78	3.68	5.74	7.78	10.27	11.17	10.55	8.29	6.89	4.73	3.36	2.62	77.86
Wilcox, Ariz.‡	....	3.59	4.73	7.29	10.10	11.24	12.20	10.64	9.03	8.10	6.70	4.67	3.41	91.70
Yuma Evaporation Station, Ariz.‡	....	3.09	3.89	5.72	7.23	8.24	8.91	10.24	9.96	7.85	5.39	3.40	2.62	76.54
Carlsbad, N. Mex.‡	....	4.50	4.50	5.51	7.45	10.12	11.05	12.88	12.00	9.50	7.00	5.75	4.50	94.76
Agriculture College, N. Mex.‡	....	2.87	4.50	7.41	9.37	11.10	11.91	11.15	9.79	8.09	5.93	3.63	2.52	88.27
Elephant Butte, N. Mex.‡	....	2.78	4.49	7.56	10.44	13.43	14.48	12.22	10.83	9.07	7.76	4.32	3.05	100.43
Lake Avalon, N. Mex.‡	....	2.84	3.26	5.49	7.49	7.87	8.70	10.13	9.60	8.59	6.49	3.17	2.42	75.55
Santa Fé Field, N. Mex.‡	....	1.53	2.13	3.96	6.15	8.51	10.14	8.93	8.06	6.55	4.81	2.55	1.93	65.25
Spur, Tex.‡	....	2.79	3.54	4.82	5.48	6.66	8.68	9.33	7.88	5.59	4.48	3.42	2.34	65.01
Hill Ranch, Tex.‡	....	2.47	3.43	5.29	6.01	6.81	7.95	8.97	8.72	6.36	4.96	3.18	2.44	66.59
Beeville, Tex.‡	....	2.69	3.26	4.26	4.79	6.21	7.35	8.14	8.14	5.89	4.72	3.06	2.17	60.68

* "Regulation of the Great Lakes," by John R. Freeman, Past-President, Am. Soc. C. E., 1926.

† *Engineering-News,* 1910.

‡ *Monthly Weather Review,* July, 1927, p. 320.

It is interesting to note that by using the insolation quantity as computed by Method No. 2 and by using the value of radiation, (B_r), given for any one of four places mentioned in Table 6, the evaporation computed for the fifth place was within 5% of the observed evaporation in each case.

Before taking up the practical applications of Equation (10) it is well to consider Table 10, which contains good evaporation records for stations in the United States. Unfortunately, these records were from tanks which were not of the same size. It is well also to bear in mind that the amount of evaporation from a small tank next to a lake is not the same as that from the larger body of water, but represents simply an index. The data in Table 10 were compiled from records presented in various places.* Equation (10) may be solved by substituting observed values of I from sources such as Table 3 and by determining values of B_r from Equation (8). Table 11 gives a comparison of the observed and computed evaporation near pyrheliometer stations.

TABLE 11.—Comparison Between Observed and Computed Evaporation.

Station.	Insolation observed at pyrheliometer stations, in inches per year, divided by 1.22.	Back radiation, in inches per year, divided by 1.22.	Computed evaporation, in inches per year.	Observed evaporation, in inches per year.
Boston, Mass	60.0	17.3	42.7	39.11
New York, N. Y	50.4	17.3	33.1	31.76
Cincinnati, Ohio	66.0	17.3	48.7	49.99
Great Lakes	49.5	17.3	32.2	27.96
Birmingham, Ala	66.1	15.0	51.1	51.3
Mitchell, Nebr	73.7	17.0	56.7	65.67
Boise, Idaho	78	17.3	60.7	77.43
Klamath, Ore	62.7	17.3	45.4	54.40
Lake Tahoe, Calif	62.7	17.3	45.4	42.2

Evaporation rates have been computed for the stations given in Table 10 with good correspondence. The results of three other computations were as follows: Roger C. Wells, of the U. S. Geological Survey, determined the evaporation of Chesapeake Bay to be 120.9 cm. per year,* while the computed amount, using his excellent data, was 124.3. McEwen by one method and Grunsky by another† determined the evaporation of Lake Mendota (near Madison, Wis., a pyrheliometer station) for a 5-month period, to be 41.45 and 45.5 cm., respectively, while the computed amount was 39.66. The evaporation of Swiss Alps lakes‡ from a 90-day period amounted to 198.3 cm., while the computed amount was 193.7.

When the value of insolation is not known from pyrheliometer records it is possible to compute the insolation, I_{ex}, incident upon the exterior of the earth's atmosphere for any latitude, and, later, to convert this into actual Insolation (I) by means of the two coefficients, ρ' and F, the mean transmission coefficient

* *Journal*, Washington Academy of Science, October 19, 1928; U. S. Geological Survey, Dept. of Interior, *Professional Paper 154C*, March 14, 1929.

† Technical Series, Scripps Institution of Oceanography, Univ. of California, Vol. 2, No. 6, September, 1929, pp. 177–306.

‡ *Monthly Weather Review*, August, 1925, p. 355.

of the atmosphere at a particular place and the clearness factor, respectively. The relation may be expressed by the integral:

$$I_{ex} = 2\,K \int_0^t = \cos \psi \, d\, t \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Integrating, after writing for $\cos \psi$ its equivalent, $\sin \lambda \sin d + \cos \lambda \cos d \cos w\, t$, Equation (11) becomes:

$$I_{ex} = (2\,K\,t \sin \lambda \sin d) + \left(\frac{2\,K}{w} \cos \lambda \cos d \sin w\,t\right) \ldots\ldots (12)$$

in which, λ = the latitude of the station; d = the sun's declination for a certain day of the year; and $w = \frac{2\,\pi}{24 \times 60}$ = the angular velocity of the earth about its axis.

TABLE 12.—EVAPORATION AS DETERMINED BY COMPUTING THE INSOLATION RECEIVED ON THE EXTERIOR OF THE EARTH'S ATMOSPHERE.

Values of λ, in degrees north latitude.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
	VALUES OF I_{ex} IN CALORIES PER SQUARE CENTIMETER PER DAY.											
52	189	305	512	743	916	993	958	818	527	387	226	161
49	229	347	548	765	929	1 003	965	837	562	425	270	198
48	241	359	557	770	932	1 000	964	838	579	441	283	211
45	286	404	596	796	945	1 008	976	861	611	478	323	255
42	322	440	623	809	943	1 004	974	870	638	510	361	291
41	336	450	634	815	951	1 002	976	871	647	521	376	291
37	392	505	678	844	963	1 032	993	896	696	572	433	367
33	449	556	717	864	963	1 003	985	908	731	619	489	424
30	487	585	735	871	960	992	977	911	799	646	519	461
29	491	600	746	875	956	990	974	913	807	651	533	476
20	618	702	714	904	947	958	851	925	856	746	642	595
Values of λ, in degrees north latitude	VALUES OF EVAPORATION, E, IN INCHES OF EVAPORATION FROM A WATER SURFACE CORRESPONDING TO THE ABOVE VALUES OF I_{ex}.											
52	3.9	5.7	10.6	14.9	18.9	19.9	19.7	16.9	10.5	8.0	4.5	3.4
49	4.7	6.5	11.3	15.3	19.2	20.1	19.9	17.3	11.2	8.8	5.4	4.1
48	5.0	6.7	11.5	15.4	19.3	20.1	19.9	17.3	11.6	9.1	5.7	4.4
45	5.9	7.5	12.3	15.9	19.5	20.2	20.2	17.8	12.2	9.9	6.5	5.3
42	6.7	8.2	12.9	16.2	19.5	20.1	20.2	18.0	12.8	10.5	17.2	6.0
41	6.9	8.4	13.1	16.3	19.6	20.1	20.2	18.0	12.9	10.8	7.5	6.3
37	8.1	9.4	14.0	16.9	19.9	20.6	20.5	18.5	13.9	11.8	8.7	7.6
33	9.3	10.4	14.8	17.3	19.9	20.1	20.4	18.8	14.6	12.8	9.8	8.8
30	10.1	10.9	15.2	17.4	19.8	19.8	20.2	18.9	16.0	13.3	10.4	9.5
29	10.3	11.2	15.4	17.5	19.8	19.8	20.1	18.9	16.1	13.4	10.7	9.8
20	12.8	13.1	14.8	18.1	19.6	19.2	19.6	19.1	17.1	15.4	12.8	12.3

The time, t, is the number of minutes from noon to sunset and the value of Abbot's solar constant, K, is 1.93 calories per sq. cm. per min. Values of

TABLE 13.—THE RATIO OF OBSERVED INSOLATION AND THE COMPUTED INSOLATION STRIKING A HORIZONTAL SURFACE ON THE EXTERIOR OF THE EARTH'S ATMOSPHERE FOR ALL STATIONS WHERE INSOLATION RECORDS HAVE BEEN KEPT.

Station.	Years observed.	Values of the Ratio, $\frac{I}{I_{ex}}$.													
		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Average monthly.	Deviation from the mean.
New York, N. Y.	1924–1929	0.34	0.42	0.40	0.45	0.39	0.36	0.40	0.31	0.38	0.38	0.30	0.31	0.37	0.038
Washington, D. C.	1924–1929	0.47	0.42	0.50	0.51	0.48	0.48	0.46	0.47	0.51	0.51	0.46	0.44	0.48	0.026
Chicago, Ill.	1924–1929	0.31	0.28	0.35	0.42	0.45	0.33	0.38	0.43	0.49	0.33	0.24	0.27	0.36	0.069
Madison, Wis.	1924–1929	0.55	0.46	0.50	0.53	0.48	0.52	0.47	0.49	0.49	0.42	0.39	0.45	0.38	0.034
Lincoln, Nebr.	1924–1929	0.64	0.53	0.58	0.57	0.49	0.56	0.56	0.55	0.57	0.56	0.54	0.57	0.56	0.023
Twin Falls, Idaho	1927–1928	0.53	0.48	0.39	0.44	0.40	0.60	0.70	0.80	0.82	0.77	0.51	0.54	0.58	0.13
Vancouver, B. C., Canada	1924	0.41	0.43	0.37	0.36	0.39	0.35	0.35	0.38	0.35	0.38	0.38	0.35	0.38	0.020
Victoria, B. C., Canada	1924	0.38	0.40	0.44	0.42	0.43	0.40	0.41	0.41	0.41	0.38	0.36	0.35	0.41	0.024
St. Johns, N. B., Canada	1924	0.46	0.52	0.46	0.42	0.42	0.42	0.44	0.45	0.46	0.41	0.43	0.39	0.44	0.025
Boat off Boston, Mass.	1924	0.42	0.47	0.50	0.47	0.47	0.47	0.44	0.44	0.47	0.50	0.46	0.43	0.46	0.020
Near Eureka, Calif.	1924	0.45	0.52	0.48	0.51	0.51	0.52	0.46	0.45	0.47	0.49	0.46	0.46	0.48	0.024
Boat off Florida	1924	0.43	0.40	0.48	0.47	0.47	0.44	0.44	0.44	0.42	0.42	0.49	0.43	0.45	0.022
Lower California, Mexico	1924	0.62	0.59	0.55	0.54	0.49	0.48	0.46	0.49	0.54	0.55	0.61	0.62	0.55	0.045
Santiago, Cuba	1924	0.54	0.58	0.54	0.52	0.48	0.52	0.49	0.49	0.51	0.52	0.52	0.52	0.52	0.018
Pasadena, Calif.	1926–1927	0.70	0.70	0.61	0.63	0.59	0.51	0.59	0.63	0.63	0.65	0.76	0.75	0.65	0.056
Fort Collins, Colo.	1926	...	...	...	...	...	...	...	...	0.57	...	...	...	...	...
San Francisco, Calif.	1927	...	...	...	...	...	...	...	0.65	...	...	...	...	...	...
San Diego, Calif.	1927	...	...	...	...	...	...	0.69	...	...	...	...	...	...	...
La Jolla, Calif.	1928–1929	...	...	...	...	...	...	0.56	...	...	...	...	...	...	...

declination, d, vary from $+ 23°$ on June 21 to $- 23°$ on December 21 of each year. The results of applying these values in Equation (12) to obtain quantitatively the insolation striking the exterior of the earth's atmosphere are given in Table 3 in "Computed Insolation." Table 12 gives additional values of exterior insolation for eleven different latitudes extending from Canada to Mexico, together with the equivalent, in inches of evaporation. These values of evaporation would represent the physical maximum because they assume a constant back radiation, no clouds, and a transmission coefficient of 1.00.

Clearness Factors

At the beginning of this paper the writer stated that there is need for a unit of cloudiness and for records of cloudiness, kept in a uniform manner. At present, this factor may be expressed as a clearness factor to be computed by Kimball's formula:*

$$F = \frac{Q}{Q_s} = 0.22 + 0.78\,(1.0 - C') \qquad (13)$$

in which, F is a clearness factor; Q is the observed radiant energy that passes through local clouds; Q_s, the observed radiant energy that passes through a clear sky; and C' the percentage of cloudiness that occurred during a certain month as given in *Monthly Weather Review* reports.

This value is useful in converting the insolation, I_{ex}, on the exterior of the earth's atmosphere to surface insolation, I, and to evaporation. In algebraic terms:

$$I = I_{ex}\,\rho'\,F \qquad (14)$$

Equation (14) demonstrates that the combined effect of relative cloudiness and the atmospheric transmission coefficient may be expressed as the ratio, $\frac{I}{I_{ex}}$. This is an important reason for accumulating insolation data. Table 13 includes values of this ratio.

Conclusions

It is possible to determine the value of insolation by three distinct methods, and the results obtained by these different methods check experimentally within 5 per cent. The six Government stations that record insolation are mostly in the eastern part of the United States and more stations in the Western States are highly desirable. Where it is not possible to get a ratio of evaporation to insolation from pyrheliometer records, it is necessary to compute the insolation on the exterior of the earth's atmosphere and correct this quantity by a transmission coefficient for the earth's atmosphere and by a clearness factor (see Equation (14)). The evaporation formula (Equation (9)) given in terms of insolation and radiation checks experimentally with observed evaporation in California, and when applied to bodies of water outside California it gives satisfactory results.

The author is indebted to Franklin Thomas, M. Am. Soc. C. E., for encouragement that led to the preparation of this paper.

* *Monthly Weather Review*, April, 1927, p. 156.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

CITY PLANNING AS RELATED TO THE SMALLER CITIES*

By E. A. Wood,† M. Am. Soc. C. E.

Synopsis

The National Conference on City Planning has urged American municipalities to establish City Planning Commissions, adopt a City Plan and "sell" it to the citizenship. This program embraces all cities, irrespective of size. It can probably be applied with greater ease and greater chances of success to the smaller city than to the larger one because there are fewer mistakes to correct and the subject-matter is in a more plastic stage.

The City Planning Commission should be established by ordinance, to furnish some degree of permanence to its existence. Its members should be selected from outstanding citizens who have sufficient leisure to attend the meetings, inspect plans, meet with interested owners, etc. The City Planning Commission should have a budget so that quarters may be acquired, a Consultant employed, and a staff of sufficient size to prepare maps, make studies, arrange meetings, give out publicity, etc. Rules should be formulated for the platting of property and a zoning ordinance should be adopted at an early date.

A Master Plan should be prepared covering not only the incorporated area of the city, but the adjacent territory as well. This should show all streets, alleys, blocks, schools, parks, parkways, streams, etc.

Popular support is always necessary for the carrying out of plans. This is particularly true in the smaller cities.

The Program in General

In adopting a comprehensive program for its work the National Conference on City Planning has urged each American municipality to do three things: First, provide a City Planning Commission and an office where the

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† Cons. Engr., Dallas, Tex.

Commission may take part in the City's administrative machinery; second, prepare and adopt an Official City Plan; and, third, secure public support for the administration of the official plan. Such a program may be adopted by any municipality irrespective of size, location, or composition. It also clearly shows that the difference in the relationship of city planning in large and small cities is one of size only; the principles are the same and legislation is equally necessary in either case. There is this difference, however, that city planning is easier for the smaller cities to undertake, is more effective, and does not cost nearly as much.

Every city should be as carefully planned as a modern railway terminal, a suspension bridge, or a central lighting plant; but the planning should be done before the city has developed. Further, it should be along regulatory lines before mistakes occur, rather than through the corrective method after errors have been permitted. Therefore, if the official plan is prepared in advance of city growth, its enforcement becomes a matter of public support, strengthened by the necessary rules and regulations to make it generally effective.

City Planning Commissions

As pointed out by the National Conference on City Planning, the establishment of a commission is the first step. Since private property and property rights are involved, a commission is better able to deal with the public than the Mayor and Board of Commissioners. The members of the Board do not have time to devote to the many problems that arise, nor are they politically free to treat the subject dispassionately. The Planning Commission should be created by a properly drawn ordinance to give the required official status. Preferably, the ordinance should be patterned after "A Standard City Planning Enabling Act", as prepared and published by the United States Department of Commerce.

This Act was prepared by an Advisory Committee, members of which have had many years of first-hand experience in coping with local planning problems, both as citizens and in connection with the leading National business, professional, and civic groups which they represent. During the three years' work required to draft the Act, the Advisory Committee made laborious research into legal problems and consulted with expert planners, members of planning commissions, municipal officials, and other interested persons throughout the country. The Act, therefore, represents the very best thought on city planning, is concise yet brief, and contains the necessary legal phraseology.

Personnel Required

The relationship of city planning to the smaller city should be real. It should create a lively interest on the part of every citizen. The membership of the City Planning Commission should be carefully selected and should be composed of outstanding citizens who can devote considerable time to the study of physical conditions on the ground as well as to the numerous meetings that will be required with different groups of individuals. The

membership should be varied, and if the plan is to be comprehensive and include all phases of city life, the public schools, park board, women's clubs, Chamber of Commerce, and retail merchants should be represented with other organizations.

Representatives from the City Government should be limited to a member of the Council and the City Engineer. The former should act as a liason agent between the Council and the Planning Commission. The latter is entitled to membership by virtue of his office; he is also charged with the execution of the plan and to some extent with the enforcement of the Commission's rules and regulations. The Mayor and City Attorney should be members *ex officio,* the Mayor by virtue of being the head of the Council, and the City Attorney by reason of his legal training.

The ordinance creating the City Planning Commission should also set up a budget to provide for the establishment of a City Planning Office and the employment of a City Planning Consultant. The former is necessary to provide working quarters for the Commission, a meeting place, and a forum for the creation of interest in the Official Plan. Public interest in the plan is of prime importance, and it is necessary that some one explain it, exhibit maps, and discuss advantages both pro and con, if public interest is to be sustained.

Many people respond by giving counter proposals, and interest is aroused and held if the work progresses expeditiously. In the smaller city, it is easier to create this interest and hold it, because the diversity of objectives common to the large city is lacking. Also, the opportunities for correct planning are more numerous in the smaller city; at least, the physical boundaries are more elastic and are more readily moulded into the Official Plan.

However, one must ever be on the alert for mistakes. A Consultant should be employed, not only to guide and direct the planning, but also to serve as mentor to the Commission and the citizens. It is very easy to make mistakes, and early enthusiasm for city planning benefits may lead to serious errors unless held in check by the Consultant.

Rules and Regulations

As a preliminary step the City Planning Commission should adopt a number of rules and regulations. The most important of these are the platting requirements and the zoning regulations. The platting of private property is one of the most fruitful sources of errors in city planning because it is so frequently left entirely to the individual owner, who fails to co-ordinate his plans with existing streets, blocks, and alleys. This is largely due to the absence of a Master Plan for the gradual development of the city. By means of certain regulations control of the platting of new additions is placed in the hands of the City Planning Commission and gives it adequate opportunity to study and adjust the platting to fit existing conditions.

In Texas, cities of 25 000 population, or more, now have the power to control platting, not only within the city's limits, but also for a distance of five miles beyond. In smaller cities, control within the city's limits may

be obtained by withholding water, sewer, or other utility connections until plats are approved by the City Planning Commissions.

Zoning, An Essential Feature

Zoning regulations deal with private property and not only restrict the use of land, but also limit the height of buildings and the density of population. They meet a popular demand for protection of home neighborhoods against invasion by undesirable uses. This popularity is shown by the number and class of cities in the United States having zoning regulations, to the extent of 583, as follows;

Number of Cities with Zoning Regulations.	Population.		
53	Less	than	1 000
144	1 000	to	5 000
102	5 000	to	10 000
103	10 000	to	25 000
79	25 000	to	50 000
47	50 000	to	100 000
55	More	than	100 000

This type of regulation is legal, not only in Texas, but in a majority of the States, because it is based on the police power of the Government which seeks to protect the health, safety, morals, and general welfare of the community. This authority was given to Texas cities by the Legislature in 1927.

The Master Plan

One of the most important duties assigned to the City Planning Commission is the preparation of the Master Plan, or Guide, for the future development of the city. It becomes an easier task after the adoption of the platting rules and the zoning ordinance because these measures afford the necessary means of control. The Master Plan should show not only the incorporated area, but the contiguous area as well, and should include all streets, highways, alleys, parks, school grounds, railways, interurbans, streams, lakes, and water-fronts, if any. The area to be included should be governed by the jurisdiction of the City Planning Commission over the platting of private property. In Texas, this would be a distance of five miles beyond the city limits, but in other States this would depend on the State law.

Other features of the Master Plan would be the continuation of the building lines and setbacks as established in the city proper by means of the zoning ordinance and the street-widening program. This is particularly desirable in the suburban business districts where traffic is heavy and it is essential to keep the thoroughfares as wide as practical.

Parks and Parkways

Parks are an essential part of the Master Plan because they provide for recreation. The price of real estate usually requires that they be acquired well in advance of development of adjacent property, otherwise the value of the land is too high. Certain cities now require sub-dividers to dedicate

10% of the area of their sub-divisions for park purposes, a process which materially assists the development of a park system.

Parkways, boulevards, and walkways should be located along creeks or ravines. This is very important in growing cities because it furnishes storm sewers made to order. Too frequently this feature is overlooked and natural drainage channels are filled to create additional lots or streets. As a result of the parkway adjacent property values are enhanced and natural beauty spots are preserved for future use in a boulevard system.

Public Support Builds the Future City

Popular support is necessary and any city plan must be "sold" to the public or the Master Plan will never be carried out. Experience in many cities bears out this statement. It is sometimes said that it is better to have "more selling and less planning". The essential point is to convince the community as to the advantages of planning for the future. Citizens will then demand a plan before they will vote bonds for unco-ordinated work. This is particularly true of the smaller city, where public funds are limited.

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PAPERS AND DISCUSSIONS

SILTING AND LIFE OF SOUTHWESTERN RESERVOIRS*

By R. G. Hemphill,† Assoc. M. Am. Soc. C. E.

Synopsis

This paper cites records of the silt load of several of the principal streams of the Southwest to indicate the importance of the silt problem in that region and the necessity for a comprehensive study of it to meet adequately water storage problems which will arise in the future. There follow descriptions of methods used, records obtained, and some tentative conclusions drawn from an investigation of silt in Texas streams, which is now entering its sixth year.

Erosion is governed by many factors, the more important of which are the amount and intensity of rainfall, and the character of topography, surface cover, and soil. The combination of these factors in the Southwest is favorable in a marked degree to heavy erosion.

Rainfall varies from an annual mean of about 30 in. at the eastern border of the semi-arid area of Texas to an annual mean of less than 5 in. in sections of Arizona and California.‡ Light rains are fairly numerous, but constitute only a small percentage of the total precipitation. As far as erosion is concerned, the significant feature of the precipitation is the large percentage of the total which occurs as torrential rains. The U. S. Weather Bureau reports 23.11 in. in 24 hours in September, 1921, at Taylor, Tex., the highest precipitation ever officially recorded in the United States for that period,§ and apparently reliable unofficial records for the same storm at points near-by are higher. Intensity rates of 4 to 6 in. per hour are not uncommon in Texas, and rates for other States of the Southwest are comparable.

Such rainstorms produce a rapid and heavy surface run-off which is very effective in surface erosion; and the quick collection of the run-off in drainage

* Presented at the meeting of the Irrigation Division, Dallas, Tex., April 25, 1929. Discussion on this paper will be closed in September, 1930, *Proceedings*.

† Irrig. Engr., U. S. Dept. of Agriculture, San Antonio, Tex.

‡ "Precipitation and Humidity," by J. B. Kincer, Section A, Pt. II. Atlas of American Agriculture, U. S. Dept. of Agriculture.

§ *Monthly Weather Review*, September, 1921, U. S. Weather Bureau.

channels produces sharp floods with high velocities which continue the processes of erosion and transportation of silt. As might be expected, the flow of the streams of the region is generally characterized by long periods of low stage and short, sharp floods of relatively great volume. The Colorado and Rio Grande, which carry spring floods from melting snow in their upper basins, must be noted as exceptions, although in the case of the Rio Grande much of the spring flood is diverted in the San Luis Valley of Colorado.

Except areas in Texas, the topography of the region is predominantly mountainous with steep slopes permitting a rapid run-off. The State of Texas includes a relatively small area of smooth, gently sloping plains and a much larger area of rolling plains with slopes sufficiently heavy to produce rapid run-off.

In general, the surface cover* of the region may be classified as sparse. Forests are limited to Central Texas where there are small scattered areas of oak and hickory and to the higher altitudes of Arizona and New Mexico where piñon, juniper, yellow pine, and fir are found. Creosote bush covers a large area in the four States and some sage-brush is also found. The remaining areas are covered by short, tall, or mesquite grass. The effectiveness of the grass cover in checking run-off and erosion has been reduced to some extent by excessive grazing.

The region contains mostly finely divided alluvial soils, in many instances extending to great depths. The tendency to erode is relatively high. While the area of land under cultivation comprises a small part of the total area, doubtless, under present conditions, such land contributes substantially to the silt yield of the region.

Amount and Effect of Silt in Streams

To the most casual observer it is apparent that these factors are effective in loading the streams of the region with silt. The problems involved were recognized at an early date. In the past forty years a number of agencies, including the U. S. Geological Survey, the U. S. Bureau of Reclamation, the U. S. Department of Agriculture, the various State experiment stations, and canal managements, have investigated the bearing of silt on some particular existing or proposed project. To cite the results of only a few of these investigations will indicate the extent and character of the problem involved.

Colorado River.—The silt load of the Colorado is of particular interest in view of the prospect of constructing an immense reservoir in its Lower Canyon Section. Records for 18 years at Yuma, Ariz., show an average annual load of suspended silt of 183 759 000 tons,† or, approximately, 0.84% by weight of the average annual discharge of 16 200 000 acre-ft. Assuming a drainage area of 225 000 sq. miles above Yuma, the average yield of silt per square mile per year was 817 tons.

By comparing records for short periods at Topock and Yuma, Ariz., and on Imperial Valley canals, and assuming that 1 cu. ft. of deposit would con-

* "Natural Vegetation," by H. L. Shantz and Raphael Zon, Section E, Pt. I, Atlas of American Agriculture, U. S. Dept. of Agriculture.

† "Silt in the Colorado River and Its Relation to Irrigation", by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., U. S. Dept. Agriculture, *Technical Bulletin No. 67*, 1928.

tain 85 lb. of dry silt, Fortier and Blaney* reached the conclusion that the total load of bed and suspended silt at the lower end of the Canyon Section of the river averages 137 000 acre-ft. per year. For the drainage area of 171 000 sq. miles above Topock, this is equivalent to a silt yield of 0.80 acre-ft. per sq. mile per year.

If this estimate holds good, the capacity of a reservoir formed by a 500-ft. dam* at either the Boulder Canyon site or the Black Canyon site will be reduced by one-half in less than seventy-five years by deposits of silt.

Gila River.—Only a few short-term records of silt in the Gila are available to confirm its local reputation of being the muddiest stream in the United States. A Board of Army Engineers in reporting on the San Carlos Project in 1914† estimated an average of 1.3% of silt by volume based on 70 lb. of dry silt in 1 cu. ft. of deposit. The equivalent by weight is 1.46 per cent.

Several silt surveys have been made in the Roosevelt Reservoir on Salt River, a tributary carrying less silt than the Gila. In 1925, Cragin‡ reported an accumulation of 101 000 acre-ft. of deposit after 20 years of service. At the average rate of silting shown by the surveys, about 160 years will be required for deposits to reduce the capacity of the reservoir by one-half.

Rio Grande.—On the Rio Grande, silt records are available for long periods at El Paso, Tex., and San Marcial, N. Mex. Follett, in his report on silt in this stream,§ combined the records for these two stations and concluded that at San Marcial, for the period from 1897 to 1912, inclusive, the annual discharge averaged 1 192 000 acre-ft., and contained suspended silt to the amount of 1.41% by weight. Relying on one sample taken under carefully selected conditions, he assumed that 1 cu. ft. of deposit of this silt in a reservoir would contain 53 lb. of dry material. Converting the weight on this basis, he arrived at an estimate of 19 739 acre-ft. of suspended silt per year at San Marcial.

That Follett used good judgment in selecting his volume-weight factor seems to be borne out by the silting of the Elephant Butte Reservoir immediately below San Marcial. Surveys made by engineers of the U. S. Bureau of Reclamation indicate that silt deposits are filling the reservoir at the rate of approximately 20 000 acre-ft. per year.‖ If this proves to be an average rate, the original capacity of 2 638 860 acre-ft. will be reduced by one-half at the end of 61 years of service.

Follett made no estimate of the silt rolled along the bottom of the stream, but stated his belief that it might possibly amount to as much as 25% of that suspended. Accepting this ratio as the maximum, the measured average annual deposit of 20 000 acre-ft. does not include more than 4 000 acre-ft. of bed silt. The remaining 16 000 acre-ft. of deposit is, therefore, left to be

* "Water Power and Flood Control of Colorado River below Green River, Utah," by E. C. LaRue, M. Am. Soc. C. E., Capacity Tables in U. S. Geological Survey *Water Supply Paper No. 556*.

† "Amount of Silt That Would Be Deposited in Reservoir at San Carlos," by D. S. Hughes, M. Am. Soc. C. E., House Doc. 791, 63d Cong., 2d Sess., San Carlos Irrigation Project, Arizona, Appendix G.

‡ Unpublished Report to U. S. Bureau of Reclamation, by C. C. Cragin, M. Am. Soc. C. E., Gen. Supt. and Chf. Engr., Salt River Valley Water Users Assoc.

§ "Silt in the Rio Grande," by W. W. Follett, Dept. of State; International Boundary Comm.; Comm. for the Equitable Distribution of Waters of the Rio Grande.

‖ "Elephant Butte Dam," by P. I. Taylor, "Dams and Control Works Constructed by the Bureau of Reclamation," U. S. Dept. of the Interior.

accounted for by the annual average of 22 785 000 tons of suspended silt found by Follett. The combination of this volume and weight of silt gives a unit value of 65 lb. of dry silt to 1 cu. ft. of deposit. It is probable that the 16-year record of suspended silt and the 10-year record of reservoir deposits did not develop true averages, but it is believed that they were accurate enough to indicate fairly that suspended silt in the Rio Grande will form deposits containing from 55 to 65 lb. per cu. ft. of dry silt.

Pecos River.—In 1904, a survey of Lake McMillan on the Pecos River near Carlsbad, N. Mex., showed that in ten years of service its capacity had been reduced 42% by silt deposits.* This was at the rate of about 1 225 acre-ft. per year. In 1916, a Board of Engineers of the Bureau of Reclamation, on the basis of 21 years of service, arrived at an estimate of an average inflow of 2 400 acre-ft. of silt.† This was 0.8% by volume of the estimated annual run-off.

Colorado River of Texas.—The silting of the lake on the Colorado River near Austin, Tex., was carefully observed by Taylor‡ from the time of the construction of the first dam. In less than 7 years of service prior to the failure of the dam, silt deposits reduced the capacity of the lake from 49 300 to 25 741 acre-ft., an average of 3 490 acre-ft. per year. At this average rate, in 14 years deposits would have equalled the total capacity of the lake. The drainage area above the dam has been estimated at 38 200 sq. miles and the annual deposit of 3 490 acre-ft., therefore, was equivalent to a silt yield of 0.09 acre-ft. per sq. mile per year.

The silting of this reservoir is convincing evidence of the necessity for a reasonable relation between the size of a reservoir on a silt-laden stream and the run-off it receives. The mean annual discharge of the Colorado at the dam is 1 900 000 acre-ft., or more than thirty-eight times the original capacity of the reservoir. Since the entire silt load of the stream, during the first years of service at least, was probably retained in the reservoir, it is not surprising that silting proceeded at the rapid rate of 7% of the capacity per year. Had the reservoir been twenty times as large, for instance, and subject to the same run-off and silt yield, silting would have proceeded at a rate of less than 0.4% per year, and approximately 280 years would have been required for the accumulation of deposits equal to the capacity of the reservoir.

Trinity River.—In 1928, Taylor made a survey of Lake Worth§ and found that in 13 years of service 13 837 acre-ft. of silt had accumulated in the reservoir. The average annual deposit, therefore, was 1 065 acre-ft., and amounted to 2.3% of the original capacity of the lake. On this basis, about 45 years would be required for deposits to equal the original capacity of the lake.

* "Water Resources of the Rio Grande Basin," by Robert Follansbee, M. Am. Soc. C. E., and H. J. Dean, U. S. Geological Survey, *Water Supply Paper No. 358.*

† "Silt Deposited in Reservoirs—Carlsbad Project," by T. E. Weymouth, D. C. Henny, R. F. Walter, Members, Am. Soc. C. E., and L. E. Foster, Esq.; unpublished report, Board of Engrs., U. S. Reclamation Service.

‡ "The Silting of the Lake at Austin," by T. U. Taylor, Univ. of Texas, *Bulletin No. 2439.*

§ "Silting of the Lake at Austin, Texas," by T. U. Taylor, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1681.

The drainage area above the dam is 1 870 sq. miles. The annual deposit of 1 065 acre-ft., therefore, indicates an average annual silt yield of 0.57 acre-ft. per sq. mile.

Need for Investigation

These examples clearly demonstrate the necessity of a careful consideration of the silt problem in connection with Southwestern reservoirs. While this region has lagged behind other sections of the country in the past, there is now evident a determination to bring about a full utilization of its water resources. In view of the erratic flow of the streams, it is certain that any considerable irrigation or power development must depend on stored water. It follows that one of the chief problems of the engineer of the future project will be to determine the silt load of the stream and to adjust the capacity of the reservoir to other requirements so as to give it the longest effective life within the range of feasible costs.

The best basis for an estimate of the silt load is a long-term record of silt measurements at the site of the reservoir. Such records have been available only in a very few instances, and when a new reservoir is proposed it is seldom practicable to delay the project several years while continuous records of the silt load are being obtained. The result is that estimates are based on short-term or fragmentary records at the site, or on records taken in another locality where conditions are assumed to be comparable.

Either procedure may lead to serious error. Under the circumstances, there can be no question of the real need for an extensive determination of the silt load of Southwestern streams, and the writer is optimistic enough to believe that such an investigation, taken in connection with the soil erosion studies now being made by the U. S. Department of Agriculture and several State agricultural experiment stations, will uncover fundamentals of general application.

Investigation in Texas

About 1923 the time seemed ripe for a general investigation of the water resources of Texas and of the means by which they might be brought to their maximum development. On the advice of the engineers and other forward-looking men of the State, the Legislature made a substantial appropriation and delegated to the State Board of Water Engineers the duty of carrying out a great part of the program. In addition to increasing the number of stream-gauging stations, and locating and surveying reservoir sites, it was considered necessary to determine the amount of silt carried by the streams of the State. This silt investigation was inaugurated in 1924 by the State Board in co-operation with the Division of Agricultural Engineering of the U. S. Department of Agriculture, with the writer in charge.

Due to limited funds, it was not possible to undertake the determination of the silt load of every stream of the State which offered suitable sites for reservoirs. The Brazos, therefore, was selected as the single basin which best represented in its various divisions the conditions found in other drainage areas. Sampling stations were established at nine gauging stations which

were either near reservoir sites or below areas comparable to other drainage basins. Routine daily sampling was begun immediately. These records determine the silt load at any point in the Brazos Drainage Basin and afford a reasonably sound basis for estimates of the loads of other streams of the State. As a check on such estimates, records are being obtained at single stations established on the Colorado, San Antonio, and Nueces Rivers.

Sampling Device.—Samples are taken in 8-oz., narrow-mouthed, round, glass bottles. The bottle is placed in a holder on a hanger to which is attached a stream-line current-meter weight varying in size from 7 to 100 lb., according to the velocity of the water to be sampled. Wired to the hanger is a coil spring to which is attached a rubber stopper and a cord by which the stopper may be pulled. The chief purpose of the spring is to counteract the force of the current on the chord and prevent the premature pulling of the stopper. The hanger is suspended by a cable, and if the current is swift enough to require a heavy weight, a windlass fastened to an A-frame is used to raise and lower it.

Method of Sampling.—The difficulty and cost of securing samples and handling them in the laboratory made it necessary to determine the minimum number which could, when averaged, be relied upon to represent accurately the average silt content of the entire cross-section of the stream. To arrive at the desired result, a large number of samples were taken throughout this cross-section for different stages at several stations, the number for each sampling ranging from 30 to 60. In each vertical, samples were taken at every one-tenth or one-fifth of the depth, at the surface, and as close to the bottom as it was possible to go without disturbing the condition of flow to an extent which would have vitiated the result. From these samples, vertical curves of silt percentage were plotted. Thus it was determined graphically that, within the limits of permissible error, a sample taken at six-tenths of the depth would give the average for the vertical.

In a similar manner the horizontal curve of silt percentage was plotted for each stage at each station, and it was found that the average of percentages at the center of the stream and at distances of one-sixth of the width from each bank would give the average percentage for the cross-section. For routine sampling, therefore, samples are taken at six-tenths of the depth, and at one-sixth, three-sixths, and five-sixths of the width of the stream. The probable error in this method is well within the limit fixed by the degree of accuracy which can be obtained in stream gaugings at stations such as are ordinarily found in Texas.

Laboratory Methods.—The three daily samples from each station are forwarded to Austin where the silt content is determined in a laboratory maintained for that purpose. The method used involves filtering out the silt, drying it, and obtaining its dry weight. The routine requires the weighing of: (1) A dried filter paper; (2) the bottle and sample; (3) the empty bottle; and (4) the dried filter paper and silt. The net weight of the sample and the net weight of the silt, obtained from these weights, give the percentage.

In order to retain the finest silt, a tight, stout filter paper is used. Drying is done in an automatic electric oven set to maintain a temperature of 110°

cent. To overcome troublesome variations in atmospheric moisture, the filter papers, before being used, are dried out at 110° cent., cooled in a desiccator, and taken out one or two at a time and weighed. By following the same procedure with the filter paper containing silt, after drying is completed, the error occasioned by the rapid absorption of atmospheric moisture is reduced to a negligible point.

When the samples show a large amount of silt, one of each set of three is first poured into a 40-in. glass tube with a uniform ¾-in. bore, and allowed to settle for one week to determine the percentage by volume for that period of settling. The sample is then passed through the regular routine to determine the percentage by weight. This gives for the same sample the percentage by weight and the percentage by volume after a week of settling.

It is recognized that volumetric percentages determined by seven days of settling in a glass tube do not directly represent the volume the silt will occupy when deposited in a reservoir. It is possible, however, that a proper reduction factor can be found and applied with little, if any, greater chance for gross error than now exists in the selection of a factor for converting a given weight of silt to the volume it will occupy when deposited in a reservoir.

Suspended Silt in Brazos River.—To indicate the character of the results being obtained, tentative data covering the load of suspended silt at four of the Brazos Basin Stations during the first three years of the investigation, are given in Table 1. Three of the stations included are on the main stream: (*a*) Mineral Wells, which is in the upper part of the basin and catches the combined flow of the principal upper tributaries; (*b*) Waco, near the middle of the basin immediately below the junction of the Bosque River; and (*c*) Rosenberg, only a short distance above the mouth of the stream. The station near Aspermont, on the Double Mountain Fork, is included as representing heavily laden branches of the stream near its source. The results involve the silt load for each day, the percentage shown by the three samples being applied to the discharge in order to determine the daily silt tonnage. The average percentages shown are based on the total tonnage and the total discharge for the period.

TABLE 1.—SUSPENDED SILT SHOWN BY PRELIMINARY COMPUTATION OF RECORDS AT FOUR STATIONS IN THE BRAZOS BASIN FOR THE THREE-YEAR PERIOD FROM OCTOBER 1, 1924, TO SEPTEMBER 30, 1927.

Station.	Drainage area, in square miles.	Annual discharge, in acre-feet.	Annual Load of Suspended Silt.		Annual yield per square mile, drainage, in tons.
			Tons.	Percentage.	
Double Mountain Fork near Aspermont, Tex.	7 980	170 000	4 942 000	2.13	619
Brazos, near Mineral Wells, Tex.	23 100	981 000	13 506 000	1.01	585
Brazos, at Waco, Tex.	28 500	1 629 000	19 054 000	0.86	669
Brazos, at Rosenberg, Tex.	44 000	5 188 000	30 528 000	0.43	694

As was to be expected, the highest average silt content—2.13%—was found at the Aspermont Station on the Double Mountain Fork, which drains an

area particularly subject to erosion. At Mineral Wells, the flow includes the discharge of other upper tributaries carrying relatively less silt and the percentage drops to 1.01. Below Mineral Wells, increasing rainfall dilutes the silt content and lowers the percentage to 0.86 at Waco and to 0.43 at Rosenberg. The percentages for 1925 were uniformly much higher than for the other two years. This is accounted for by the fact that 1925 was a year of unprecedented drouth, during which only two floods occurred, one in May and one in September. These floods, following dry periods of more than three months, carried extraordinarily high percentages of silt. For this reason it is believed that average percentages shown by records over a long period will be appreciably less than those shown by the three-year period.

It is interesting to note the rather close agreement in the average yield of silt per square mile of drainage above the four stations. The lowest average—585 tons for the area above Mineral Wells—is more than 84% of the highest average of 694 tons for the area above Rosenberg. By segregating the different areas, however, it is found that for the area which drains into the river between Mineral Wells and Waco the average yield of silt was 1 027 tons per sq. mile; and for the area between Waco and Rosenberg, 740 tons per sq. mile. The unusually high yield between Mineral Wells and Waco is believed to have been due partly to sharp floods in the Bosque River, but the general increase of yield down stream is probably due chiefly to increasing rainfall and increasing acreage under cultivation.

Solely for the sake of comparison, the volume of the silt load at each of the stations has been computed on the basis of 70 lb. of silt to 1 cu. ft. of deposit. At this rate, the suspended silt at Aspermont amounted to, roughly, 3 000 acre-ft., or 0.41 acre-ft. per sq. mile of drainage; at Mineral Wells, 9 000 acre-ft., or 0.38 acre-ft. per sq. mile; at Waco, 12 000 acre-ft., or 0.44 acre-ft. per sq. mile; and, at Rosenberg, 20 000 acre-ft., or 0.45 acre-ft. per sq. mile.

Reservoir Surveys.—An important phase of the investigation in Texas has been the actual measurement of deposits of silt in reservoirs. It was the intention to find a reservoir where conditions were such that the silt discharge of the stream could be measured for a period of years as it entered the reservoir and where at suitable intervals measurements could be made of the actual deposit in the reservoir, thus establishing for one set of conditions an absolute relation between measured silt in suspension and the corresponding deposit in the reservoir. Unfortunately, the required conditions could not be found and thus far this phase of the work has been confined to surveys of the actual deposits in Medina Lake, near San Antonio, Tex., Lake Worth, near Fort Worth, Tex., and Lake Kemp, near Wichita Falls, Tex.

The survey of Medina Lake showed that in 13 years of service silt deposits to the amount of 2 692 acre-ft. had accumulated. The yearly average was 207 acre-ft., which was equivalent to a yield of 0.35 acre-ft. per sq. mile of the approximately 600 sq. miles of drainage area above the dam. The capacity of the reservoir is 254 000 acre-ft. and, at the rate shown by the survey, deposits will equal the capacity in 1 225 years.

The survey of Lake Worth was made in 1925 and indicated that silting was taking place at the rate of approximately 1 000 acre-ft. per year. At this rate, deposits would equal the capacity of the reservoir in 47 years. This result was checked very closely by Taylor's survey made in 1928. The yearly average deposit of 1 000 acre-ft. is equivalent to a yield of 0.54 acre-ft. per sq. mile of drainage area above the dam. It is interesting to note that if the average silt yield, in tons per square mile, from the area above this reservoir is comparable to the yield from the near-by Brazos drainage, as shown by records at Glenrose, the deposit in the reservoir contains less than 60 lb. of silt per cu. ft.

A complete survey of deposits in Lake Kemp was attempted in its second year of service; but brush left in the basin was so heavy in certain sections that lines could not be put through. It was not considered advisable to make an estimate of the total silt deposit based partly on such random soundings as could be made in these sections. The maximum depth of silt, 4 ft., was found at the upper end of the lake, but fine silt to a depth of not less than 2.5 ft. was found in the old river channel throughout the reservoir.

Characteristic delta formations, largely of coarse material, were found at the head of these lakes. Below the delta areas, the deposits consisted of fine material and were so distributed as to lead to the belief that the finest silt when deposited will flow along the bottom of the reservoir until blocked by the dam. There seems to be no other explanation for the fact that the thickness of the layer of fine material in the case of each reservoir was little less at the dam than it was 8 or 10 miles above. If this condition proves to be general it indicates a possibility of relieving the reservoir of some silt by making a practice of drawing off water through the lowest outlet gate.

The Weight of Deposited Silt

The most satisfactory and, in fact, the only practical method of determining the amount of silt in samples of river water is by weight; but since the space the silt will occupy in a reservoir is the important consideration, it is necessary to fix a relation between the weight of the silt and its volume when deposited. There is a wide difference of opinion in regard to this relation, and values have ranged from 120 lb. per cu. ft., used by Humphreys and Abbot* in estimating the volume of silt carried in suspension by the Mississippi River, to 53 lb. per cu. ft., used by Follett in connection with the Elephant Butte Reservoir. It seems clear that the volume-weight factor of a deposit will depend chiefly on the proportions of fine and coarse material, the degree of separation which takes place on deposit, the depth of deposit, and whether it is subject to exposure and consequent drying, consolidation, and mixing by the scouring action of later inflow. The selection of a factor for a given reservoir should be governed by a consideration of all these elements.

The proportions of the different sizes of silt carried by streams vary greatly, and under ordinary conditions there is a substantial separation into different grades when the material is deposited. Silt-laden water entering the quiet

* Report upon the Physics and Hydraulics of the Mississippi River, by A. G. Humphreys and H. L. Abbot, Bureau of Topographic Engrs., U. S. War Dept., 1861.

water of a reservoir tends to continue its course by pushing aside and upward the clear water of the reservoir, and its velocity is reduced more or less gradually. This action is responsible for the rather high degree of separation which has been noted in actual deposits in reservoirs. A fluctuating water level, drying deposits, and the scouring action of a fluctuating inflow are mainly responsible for such mixing of different sizes of silt as occurs.

Some writers have assumed that increasing hydrostatic pressure may be counted on to compact deposits, but there seems to be little ground for the assumption. The writer believes that, unless exposed, deposits will be compacted to a significant degree only by the weight of the silt itself and the overburden of new deposit. In the surveys of Texas reservoirs, no difference in density could be detected in bottom deposits of fine silt under depths of water ranging from 5 to 125 ft. Deposits of fine silt will compact very little unless exposed. Those in the lake at Austin, Tex., for instance, are so loose that a 1-in. rod can be forced into them with one hand to a depth of 10 to 12 ft. When exposed to air and sunlight, however, they dry out, shrink heavily, and crack, and on being submerged again they do not swell to their original volume.

As a guide in determining the relation of volume to weight under various conditions, a large number of sample cubes were taken from deposits in place in Texas reservoirs and the dry weight of the silt was determined. Exposed bars of coarse silt, such as is found at the head of reservoirs, averaged 92 lb. of dry material to 1 cu. ft. of deposit, while finer silt in much the same location averaged 82 lb. Samples from the surface of deposits near the middle of reservoirs averaged 55 lb. of dry material to each cubic foot. Samples of the finest material taken from submerged deposits in old river channels in reservoirs averaged 31 lb. of dry material to 1 cu. ft.

Bed Silt

In addition to the silt carried in suspension, a certain quantity is moved along the bottom of the stream; but no investigator has yet devised a satisfactory method of measuring its amount. Estimates vary widely. Humphreys and Abbot, from a study of the movement of bars at the mouth of the Mississippi, estimated the bed load to be 11% of the suspended load by volume*; but in arriving at the volume of suspended silt, a volume-weight factor of 120 lb. per cu. ft. was used. It is now generally recognized that a much lower factor should be adopted, in which case the percentage of bed silt would be substantially lower. By the use of traps in the San Carlos River of Costa Rica, A. P. Davis, Past-President, Am. Soc. C. E., arrived at percentages of 5.2, 1.7, and 7.1 for June, July, and August.† By comparing records at Yuma and Topock, Fortier and Blaney estimated the bed load of the Colorado at Yuma as 25% of the suspended load.‡

Until more light is thrown on this question, a degree of uncertainty must attach to estimates based on suspended silt alone. Simultaneous vertical curves

* Report upon the Physics and Hydraulics of the Mississippi River, by A. G. Humphreys and H. L. Abbot, Bureau of Topographic Engrs., U. S. War Dept., 1861.

† Noted in "Irrigation Practice and Engineering": Vol. II, "Conveyance of Water," by B. A. Etcheverry, M. Am. Soc. C. E., p. 87.

‡ "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier and Harry F. Blaney, U. S. Dept. of Agriculture, *Technical Bulletin No. 67*, 1928.

of silt content and velocity in Texas streams incline the writer to believe, however, that the amount of silt transported as a true bed load comprises a much smaller percentage of the total load than is generally assumed.

Removing Silt from Reservoirs

Various plans for the removal of silt from reservoirs have been proposed, but a satisfactory and feasible method is yet to be found. One plan calls for an auxiliary basin at the head of the reservoir to catch the silt and a conduit through which it could be flushed to a point below the reservoir. Such a system, no doubt, would work with some degree of effectiveness, but its cost would be prohibitive. Another method involves stirring up the silt with a jet and allowing it to pass out with water drawn off, but this plan does not work satisfactorily even in the smallest reservoirs.

Dredging will naturally come to mind as a method with possibilities but under present conditions the cost would be prohibitive. As favorable sites become scarcer and the cost of this method decreases through improvements in methods and machinery, it is possible that dredging may in time become economically feasible.

Some small reservoirs have been partly cleaned by draining off the stored water and permitting the flow of the stream to scour out the deposit. It is doubtful whether this method could be used with any degree of success with larger reservoirs. The stream will cut a narrow channel through the deposits, and scouring action will cease before any considerable proportion of the deposit has passed out of the reservoir. There would be a better chance for the successful use of this plan if satisfactory methods could be devised to direct the flow of water to any point desired. Another objection to sluicing is that where water for irrigation is valuable enough to warrant the construction of extensive storage works it is too scarce and valuable to be used in clearing a partly silted reservoir. At present, the only feasible method of remedying the situation created by a silted reservoir is to secure additional storage capacity by increasing the height of the dam or by building a new reservoir.

Prevention of Silting

Surface soils and banks of channels under the erosive action of flowing water are the two chief sources of silt. The problem of preventing the silting of reservoirs, therefore, will involve either the elimination of, or a reduction in the amount of, silt thus produced, or its removal from the stream above the reservoir.

Under present conditions the outlook for effective measures to these ends is not particularly encouraging. Forest and grass cover on public lands can be maintained, but the private owner is allowed to clear his woodland, over-graze his pasture land, and permit his cultivated land to wash and add to the silt load of the streams. There is, however, a growing tendency among farmers to terrace their lands, to adopt contour plowing, and to check gullying; and as these practices become more widespread there must result eventually a substantial reduction in the amount of silt which reaches the streams.

For reducing the quantity of silt developed by the erosion of banks, dependence must be placed chiefly on flood control, which, by holding the extreme flow of the stream down to moderate stages, will eliminate the high velocities mainly responsible for excessive erosion. Barriers of brush or other material in the smaller channels and lines of living brush across the flood-plains of larger channels, by decreasing velocities and thus encouraging deposits, should relieve these channels of a part of their load of silt. The effectiveness of a dense growth of salt cedars in desilting the Pecos at Lake McMillan is reported by Walter.* This growth immediately above the lake dates from about 1918 and is responsible for a reduction in the rate of silting of the reservoir from an average of 1 930 acre-ft. per year from 1894 to 1915 to an average of 350 acre-ft. per year from 1915 to 1925.

There is a possibility of desilting a stream above a reservoir by diverting the silt-laden water through canals leading out over the flood-plain and permitting the silt to deposit in natural or artificial basins before the water is returned to the channel of the stream. Such a system would be costly, but might prove feasible in particular cases.

The problem of preventing the silting of reservoirs is one which should be handled by an organization to deal with a stream as a whole. Each of the desilting works or reservoirs that such an organization would construct, would be required to absorb only a part of the silt burden of the stream, and costs could be spread on an equitable basis over all those benefited. Otherwise, each reservoir would profit without cost from any desilting works or reservoirs in the stream above it.

Estimating the Effective Life of a Reservoir

In a study involving the effective life of a storage reservoir the first step is to determine the average silt load carried by the stream at the site. Then a consideration of the fineness of the silt, the manner in which it may be expected to be deposited, and the exposure and scouring to which the deposits will be subjected, will permit a conclusion as to the volume the yearly load of silt will occupy. The problem then becomes one of adjusting the size of the reservoir to meet the requirements for a long life at a feasible cost. In this connection, attention should be called to deposits above the flow line, the amount of silt which may pass over the spillway, the amount which may be removed through outlet gates, and the return seepage.

When silt-laden water strikes the back-water of the reservoir its velocity is reduced, and the coarser material is deposited immediately. This deposit continues to grow and extend up stream. When conditions are favorable, it may build up to the point where a substantial volume of deposit, which otherwise would occupy space in the reservoir, is above the flow line.

Samples taken on the spillway of the Austin Dam and for several months on the spillway of Lake Worth indicate that until silting has gone so far as to produce channel conditions in the reservoir, the amount of silt which passes over the spillway is negligible. The reservoir at Austin is so filled with silt that channel conditions prevail and the percentage of silt in the water passing over

* *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1722–1723.

the spillway differs very little from that in the flow above the reservoir. On the other hand, at Lake Worth where silting is only moderately advanced and channel conditions do not obtain, samples taken on the spillway after the inflow of heavy floods carrying large quantities of silt rarely showed a silt content in excess of 0.02% by weight.

It is well known that in some instances when a reservoir is lowered at a relatively rapid rate water from the saturated sides and bottom seeps back into the reservoir and may amount to a substantial quantity. It is believed that deposits of the very fine material which constitutes the greater part of the silt load of Texas streams will give up water at a very slow rate, probably not fast enough to offset evaporation at its surface. These deposits should not be depended upon as an aid in tiding over critical periods.

That water drawn off through low reservoir outlets will carry some silt with it, is certain, but the amount is doubtful. The scouring effect of the water drawn into the outlet will extend up stream only a short distance, but if the finest silt flows along the bottom of the reservoir, as was indicated at Medina Lake, Lake Worth, and Lake Kemp, there is a possibility that a substantial amount of this fine silt will reach the area affected by the gate and be carried out.

Administration

The investigation of silt in Texas streams is being done under a co-operative agreement between the Texas State Board of Water Engineers and the Division of Agricultural Engineering of the U. S. Department of Agriculture. S. H. McCrory, M. Am. Soc. C. E., is Chief of the Division and W. W. McLaughlin is Associate Chief in charge of the Division's work in the West. The Board of Water Engineers includes John A. Norris, *Chairman*, and C. S. Clark, Members, Am. Soc. C. E., and A. H. Dunlap, Assoc. M. Am. Soc. C. E. Records of stream flow are being furnished by C. E. Ellsworth, Assoc. M. Am. Soc. C. E., District Engineer, U. S. Geological Survey, who is in charge of the co-operative stream measurement work of the Survey and the Board of Water Engineers. The writer is in charge of the investigation. O. A. Faris, M. Am. Soc. C. E., is Chief Assistant, and is to be credited with handling the greater part of the field and office work of the project.

the spillway differs very little from that in the flow above the reservoir. On the other hand, at Lake Worth where silting is only moderately advanced and ultimate conditions are not obtained, samples taken on the spillway after the inflow of heavy floods carrying large quantities of silt rarely showed a silt content in excess of 0.02%, by weight.

It is well known that in some instances when a reservoir is lowered at a relatively rapid rate water from the saturated sides and bottom seeps back into the reservoir and may amount to a substantial quantity. It is believed that deposits of the very fine material which constitutes the greater part of the silt load of Texas streams will give up water at a very slow rate, probably not fast enough to affect evaporation at its surface. These deposits should not be depended upon as an aid in future conservational periods.

That water drawn off through low reservoir outlets will carry some silt with it is certain, but the amount is doubtful. The scouring effect of the water drawn into the outlet will extend up stream only a short distance, but if the sand, silt flows along the bottom of the reservoir, as was indicated at Medina Valley, Lake Worth, and Lake Kemp, there is a possibility that a substantial amount of this fine silt will reach the area affected by the gate and be carried out.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ENGINEERING FOUNDATION COMMITTEE ON ARCH DAM INVESTIGATION

ARCH DAM INVESTIGATION

Discussion*

By Fredrik Vogt, Assoc. M. Am. Soc. C. E.

Fredrik Vogt,† Assoc. M. Am. Soc. C. E. (by letter).‡—The increased use of single-span arch dams evident from Tables 9 to 14§ is very striking. The first multiple-arch dam in the United States was constructed in 1908, and was preceded by only one dam in India and one in New South Wales. In the following decennium a great number of multiple-arch dams were constructed, but only a few have been built in later years. The first single-span arch dam in the United States was constructed in 1883, but otherwise only a few such dams were built before 1910. Since then, however, a rapidly increasing number has been constructed each year. Even if all dam sites are not similar, this tendency is evident from Table 53, which shows the number of different types of arch dams in the United States on which construction work has been started in the last four 5-year periods. This table was compiled from Tables 9 and 13; at least ten more single-span arch dams, not included in these tables, have been constructed in the United States.

A complete comparison should also include gravity dams and Ambursen dams which represent the other types usually constructed on rock foundation. The number of gravity dams constructed during these years is about uniform,‖ and the number of new Ambursen dams is very small compared with earlier periods.

* Discussion on the Report of the Committee of Engineering Foundation on Arch Dam Investigation, continued from August, 1929, *Proceedings*.

† Docent, Norges Tekniske Hoiskole, Trondhjem, Norway.

‡ Received by the Secretary, January 14, 1930.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, pp. 24 to 41, inclusive.

‖ *Western Construction News*, April 10, 1927, p. 34.

This striking tendency toward the use of single-span arch dams is the logical consequence of experience with the different types of dams, and, therefore, should be emphasized. It ought in this connection to be stated that no actual single-span arch dam has failed to date (January, 1930) except at the Moyie River and Lanier Lake Dams, where the abutments failed. Despite these abutment failures the arches remained standing and they give striking evidence of resistance of the arched structure itself.

TABLE 53.—STUDY OF THE NUMBER OF DIFFERENT TYPES OF ARCH DAMS IN THE UNITED STATES

Year.	Multiple-arch.	Multiple-dome.	Single-span arch dams.
1908 to 1912	7	....	5
1913 to 1917	15	....	8
1918 to 1922	3	....	10
1923 to 1927	5	1	24

For the multiple-arch dams the development has been toward increased span between the buttresses, that is, from very thin sections toward more solid types. In the earlier period, spans of 20 ft. were common and 50 ft. exceptional in the United States. At present, spans of 50 to 60 ft. are most commonly used. The Coolidge (multiple-dome) Dam, with 180 ft. span, shows a further development toward more solid construction. Fortunately, this tendency seems not only to be a logical consequence of the experience with Gem Lake Dam and some others in cold climates, but, also, it seems to be based on a reduction in first cost.

The construction also of single-span arch dams in rather wide sites, as, for instance, the Gibson Dam, in Montana, and for the greatest heights, as the Pacoima Dam, in California, and the Diablo Canyon Dam, in Washington, is made possible only by the expensive scientific investigations carried out in later years, and Engineering Foundation is to be congratulated on the highly valuable contributions furnished in this report.

It would be desirable if Engineering Foundation could extend its service also to the investigation of gravity dams, for which many problems still seem to be unsolved. The shrinkage problem is common for all concrete dams, but shrinkage in the interior of thick dams can be investigated more easily on straight gravity dams than on curved dams for which the arching disturbs the analysis.

In order to increase the deformations and facilitate the testing work, the Stevenson Creek Test Dam was purposely constructed thinner and flatter than is considered good engineering practice. Consequently, a number of cracks were developed in the structure when loaded, and the difficulties in the interpretation of the tests were obviously increased by the cracking. With the experimental ability demonstrated in the tests, it seems as if it should have been possible to undertake tests on a structure approaching more nearly the nature of practical dams even if the deformation then had been smaller. The interpretation of such tests would have been easier, and they would have had more direct bearing on design.

The Stevenson Creek Test Dam was constructed with 5-ft. layers, usually with a 4-day interval between pourings. The dam cracked away from the abutments* during the construction period because it was not provided with contraction joints, and in some places the cracks were developed within two days after pouring. If one layer is cracked away from the abutment before the next layer is poured, the opening of the abutment crack must necessarily be irregular, that is, greater just below the horizontal construction joint than just above it, the difference in opening being equal to the opening of the first crack. The thrust due to water load must then also be irregularly distributed along the abutment joint even if the crack is closed entirely by compression. To a large extent the thrust must be concentrated at those places where the initial crack is narrowest.

This is probably the explanation of the irregularities in the measured horizontal strain near the abutments, particularly on the left side, as shown in Figs. 94, 96, and 98.† For instance, for full load the horizontal down-stream strain at Elevation 25 is measured as about twice the strain at Elevation 20, and to one and one-half times the strain at Elevation 30. This gives a much more irregular distribution of stress than otherwise could be expected, as can be seen by plotting diagrams for the strain. For part load the irregularity is still greater. At some distance from the abutments such irregularities, of course, are smoothed out. It is difficult to determine how much the maximum stresses have been increased by these irregularities. By plotting the horizontal strain at the abutments in diagrams and drawing smooth curves the writer has estimated the increase to be 40%, but of course the number is very uncertain.

A similar concentration of the thrust and increase of the maximum stresses can always be expected in dams where irregular cracking is not avoided by the use of closely spaced contraction joints extending from the crest to the very foundation. The joints must be grouted properly if concentration of stresses is to be avoided.

The irregularities in the strain may also affect the analysis of division of load on the arches and cantilevers in the Test Dam. A fairly uniform distribution of strain along the abutments would probably have given somewhat higher down-stream stresses near the abutment for the arch element at Elevation 30 and, in this way would improve the agreement between Professor Cain's formulas for uniform arch load and the test results.‡

At least at higher elevations, the abutment cracks must have been rather open for the unloaded dam, and closed more or less by loading according to the deflection tests. Let A equal the area of deflection curve plotted from the axis of an arch element; r, the radius to this axis; and U, the area of the diagram for compressive strain at the axis plotted from this. Then, the sum of the tangential displacements of the dam at the right and left abutments is:

$$\Delta\, t = \left(\frac{A}{r} - U\right) \qquad (150)$$

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, pp. 112–116.

† *Loc. cit.*, pp. 138–139.

‡ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1241.

Equation (150) should be corrected for cracks in the dam itself. Fig. 118* gives the average direct strain horizontally for full load from which U can be found by multiplication with the length. Figs. 84† and 86‡ give the deflection curve from which A can be integrated. For the crest Elevation 60, Fig. 82§ indicates practically zero movement of the bed-rock, and Equation (150) gives a tangential displacement of 0.025 in. as an average for the two ends of the dam. According to Fig. 77‖ the correction for the vertical crack at the center line will be unimportant. Since this movement is not reproduced in the bed-rock, there must have been a crack at least as wide, which was closed by the load. The cracks are described by Professor Slater,¶ but no measurements of change in width due to loading can be found recorded in the report. The opening of the cracks as computed here is in fair agreement with the total strains before loading as recorded in Fig. 74.**

That these cracks are of importance for arch action is obvious. If, for instance, the area, A, of the deflection curve was unchanged, and the tangential displacements at the ends of the dam were limited to the measured movements in the abutments, the average strain and the average thrust at Elevation 60 would have been about three times the measured value. These cracks have probably also caused the greatest part of the difference at the crest between the measured deflection and the deflection computed in the design of the dam and shown in Fig. 126.††

It is regrettable that no attempts were made to measure tangential displacements in the Stevenson Creek Test Dam even if the stiffness against such displacements was unimportant in this particular case owing to the abutment cracks. The simplification used by Professor Westergaard by setting $P_{xy} = 0$ (Equation (61)‡‡) means as a consequence that this stiffness is neglected, which places a certain limitation on the application of his analysis. For some types of dams these deformations are of importance.§§

In the analysis of the cantilevers Professor Slater‖‖ wisely states that for the lower part the results are "very doubtful". The analysis of the division of loads on arch and cantilevers in a cracked dam is certainly very difficult. In the results given in Fig. 126 the most striking point is the large bending moment which is found by the analysis at the base of the cracked crown cantilever. An idea of the depth of the crack at the base can be obtained by combining the measured opening with the slope of the deflection curve. For full load the opening is measured as 0.052 in. and the average slope for the lower 5 ft., according to Fig. 93,¶¶ is equal to 0.00072. This would give a depth

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 166.

† *Loc. cit.*, p. 131.

‡ *Loc. cit.*, p. 133.

§ *Loc. cit.*, p. 128.

‖ *Loc. cit.*, p. 123.

¶ *Loc. cit.*, p. 114.

** *Loc. cit.*, p. 116.

†† *Loc. cit.*, p. 174.

‡‡ *Loc. cit.*, p. 107.

§§ *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1272.

‖‖ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 179.

¶¶ *Loc. cit.*, p. 137.

of $\dfrac{0.052}{0.00072} = 72$ in., or 6 ft. At the very base the slope might be somewhat less, and that part of it which is caused by shear should also have been subtracted. These corrections will give still greater depths. However, some complications will also arise from the deformation in the rock, and without more direct measurements it may be difficult to find the depth of the crack very accurately. It can only be said that with the opening measured, the crack must be rather deep; perhaps three-fourths of the total thickness at the base or perhaps only one-half the thickness.

The writer has tried a number of different methods of obtaining an estimate of the moment transmitted at the base of the cantilever, using all the information given in the report. The measurements that have a direct bearing on this subject are:

(*a*) Direct measurement of the up-stream opening of the crack (see Fig. 76*).

(*b*) Telemeter readings.

(*c*) Down-stream strain measurements (see Figs. 27† and 12(*b*)‡).

(*d*) Clinometer and level-bar measurements (see Fig. 93§).

Measurement (*a*).—The full load opening in the case under discussion was found equal to 0.052 in.

Measurement (*b*).—One telemeter, which was one-half in rock and one-half in concrete, was splayed out by cracking.‖ The strains at higher elevations are shown in Figs. 134¶ and 135.¶ The vertical compression up stream at Elevation 4.0 is striking, and even at Elevation 10 the measurements indicate that the customary formula cannot be used for computing the bending moment from the down-stream stress only.

Measurement (*c*).—The computed moment corresponds to the measured strain if the section is not cracked (see Fig. 126). However, if the cracked section is exposed to such moments, the strain would be several times greater than that measured at the base.

Measurement (*d*).—The level-bar at the base (see Section 29**) gives the slope near the down-stream face only since the cracked section certainly does not remain plane after loading.

Including dead load and eventual uplift in the crack, it is difficult to understand how moments greater than one-fourth to one-half of the moment given in Fig. 126 can have been transmitted to the base, depending on the active thickness of the cantilever. Such a change, of course, will mean a radical change in all the cantilever analyses.

The formula for bending moment given in Table 24†† is only correct if there is no horizontal strain from bending (see Equation (28)‡‡); that is, the correction for lateral deformation is not correct.

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* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 121.

† *Loc. cit.*, p. 80.

‡ *Loc. cit.*, p. 63.

§ *Loc. cit.*, p. 137.

‖ *Loc. cit.*, p. 120.

¶ *Loc. cit.*, p. 184.

** *Loc. cit.*, p. 82.

†† *Loc. cit.*, p. 178.

‡‡ *Loc. cit.*, p. 162.

With the dam under full load, horizontal cracks were developed at the construction joints at Elevations 30 and 35,* and they were discovered after two days' loading. It is not illogical to assume that the concrete yielded more than was computed by the bending formula at the time of the tests, since open cracks were discovered shortly thereafter, and such a yielding will affect the cantilever analysis considerably.

In Equation (50)† Professor Westergaard gives the correct formula for the twisting moment. In order to check this Professor Slater's value given in Equation (19)‡ must be multiplied by $(1 - \mu)$. The "torsional load" is twice the cross-derivative of the twisting moment, as determined by Equation (41).§ It seems to the writer as if, in Table 22,‖ Professor Slater only gives a torsional load directly equal to the cross-derivative of the twisting moment, that is, a torsional load carried one way, which must be multiplied by 2 in order to give the total torsional load in arch plus cantilever.¶ In any case, the formulas developed are only valid as long as the thickness is constant; they are not valid for the lower part of the dam, since Equation (20),‡ developed from Equation (19), is only valid with that limitation. As soon as the thickness, t, is variable in either the x- or the y-direction, terms including the derivatives of the moment of inertia must be added.

Aside from these few critical remarks the writer has studied these reports with great pleasure and profit. He has of course been particularly interested in the check of his own formulas for yielding of the foundation as given in Fig. 82. The suggestion has been made that the spreading of the abutments could have been caused by the direct pressure of the water on the sides of the canyon. The writer, therefore, has tried to check that phase of the problem, and by using the same formulas he finds that this direct pressure can hardly have produced more than 5 to 10% of the spreading, and that this is therefore produced mainly by the thrust from the arch, as assumed by Professor Slater. A correction with these percentages would increase the computed spreading and improve the check. For an irregularly shaped canyon only approximate formulas for the yielding can be given, and the writer is well satisfied with the check obtained.

Of the many valuable investigations given by Professor Davis,** the writer considers that the tests on flow of concrete have the greatest bearing on the problem. Since dams will not be exposed to loads at early ages, the tests on concrete 3 months old are of the greatest importance. These show quite consistently that the flow is almost proportional to the stress. The ratio, $\frac{\text{unit flow}}{\text{unit stress}}$, at 3 months can be computed from Table 28 as equal to 0.23, 0.27, and 0.29 for wet concrete, and 0.70, 0.70, and 0.76 for concrete in air of 70% humidity

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 192.
† *Loc. cit.*, p. 242.
‡ *Loc. cit.*, p. 151.
§ *Loc. cit.*, p. 239.
‖ *Loc. cit.*, p. 153.
¶ *Loc. cit.*, pp. 150-153.
** *Loc. cit.*, p. 199 *et seq.*

(in millionths for stresses of 600, 900, and 1 200 lb. per sq. in., respectively). The total deformation (initial + flow) plotted in a stress-strain diagram gives fairly straight lines. If the maximum compressive stresses due to load should be released appreciably by the flow, the latter would have to increase very much faster than in proportion to the stress. This not being the case, no appreciable release can be expected for stresses within the limits used in these tests.

The stresses due to slow temperature changes are proportional to the modulus of elasticity computed from the total deformation, and are, therefore, much reduced by the flowing. This is especially true for dry concrete; the total deformation in air of 70% humidity is about three times the initial deformation on 3-month samples. In thick dams the concrete in the interior can be expected to be rather wet all the year round, and the total deformation for wet concrete at the same age is only about 1.7 times the initial deformation.

The tests on volumetric changes are valuable for slender structures, but do not have much bearing on thick dams since the conditions in the interior are different from conditions on the test samples. Only direct measurements on a great number of commercial dams can give the desired information on the shrinkage, and such measurements should be taken both at the surface and in the interior.

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The tests on volumetric changes are valuable for regular structures, but are not [illegible] much [illegible] on thick dams since the conditions in the interior are different from conditions on the test samples. Only direct measurements on a great number of monolithic dams can give the desired information on shrinkage, and such measurements should be taken both at the surface and in the interior.

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PAPERS AND DISCUSSIONS

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EXPERIMENTS TO DETERMINE RATE OF EVAPORATION FROM SATURATED SOILS AND RIVER-BED SANDS

Discussion*

By Messrs. Ivan E. Houk and Ralph L. Parshall†

Ivan E. Houk,‡ M. Am. Soc. C. E. (by letter).§—This paper is valuable to hydrologic engineers for two reasons: First, it presents carefully observed data on the quantities of water evaporated from different soils, with different depths to ground-water; and, second, it describes an automatic method of delivering easily measured quantities of water to soil tanks, as needed to supply evaporation losses, without materially changing the level of the ground-water surface.

It is to be regretted that the experiments did not include soil tanks with ground-water surfaces maintained at greater depths than 12 in., especially in the case of the fine sand and loam soils where the 1927 ratios of soil evaporation to free water-surface evaporation for the 12-in. depths varied from 50 to 100 per cent. Depths to ground-water along sandy river channels may vary from zero to several feet. Consequently, engineers engaged in hydrologic studies of such areas need to know the relations between rates of soil evaporation and depths to ground-water for the entire range of depth in which any appreciable quantities of soil moisture are raised to the ground surface by capillary action. Investigations of soil evaporation made thus far seem to indicate that the limiting depth is about 4 ft. in the case of bare sandy or sandy loam soils.

The advisability of using covers to protect the tanks from rain in conducting soil evaporation measurements is questionable. The installation of the covers when the rain begins, their removal when the rain ceases, and the proper adjustment of the evaporation records when some rain unavoidably falls on the soil tanks is probably much more troublesome, as well as more uncertain, than the measurement of total rainfall in a standard rain gauge and the correction of total measured evaporation by total measured precipitation. More-

* Discussion of the paper by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Authors' closure.

‡ Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

§ Received by the Secretary, January 15, 1930.

over, the latter method would furnish data more comparable with actual field evaporation. Nature does not provide covers when the rain begins. The engineer has more need for data on actual rates of soil evaporation immediately following rainfall occurrence than on rates of soil evaporation under water-tight covers. For this reason the data in Table 4* are valuable. Conditions of air movement and radiation comparable with those existing under the covers would occur only in the densest jungles of the tropical regions. In most forests the conditions of air movement and radiation at the surface of the ground would be intermediate between those existing under covers and in the open. In this connection the writer would like to know how the average rates of evaporation given in Table 3† were derived. Were they computed for the total times between limiting dates, or for the total times reduced by the intervals during which the covers were in place? The writer would also like to see some photographs showing the general arrangement of the station and typical details of installation.

During the fall of 1926, the U. S. Bureau of Reclamation, in co-operation with the Middle Rio Grande Conservancy District and the U. S. Weather Bureau established a soil evaporation station near Los Griegos, N. Mex., about four miles northwest of Albuquerque. The station was designed for use in studying evaporation from the low undrained bottom-lands and river sandbars along the Middle Rio Grande Valley. It was first equipped with a rain gauge, anemometer, sling psychrometer, maximum and minimum thermometers, two water-surface evaporation pans, two sand pans, and three salt-grass sod pans. Two additional soil pans were added after the station had been operated one year. Fig. 6 shows the arrangement of the equipment and the topography in the vicinity of the station, and Table 7 gives some brief notes regarding the depth and nature of the pans.

TABLE 7.—DESCRIPTION OF PANS SHOWN IN FIG. 6.

Pan No.	Depth, in inches.	Date of installation.	Contents.
1	24	September, 1926	Water.
2	10	September, 1926	Class A, U. S. Weather Bureau.
3	48	September, 1926	Sand, no vegetation.
4	48	September, 1926	Soil, salt-grass sod.
5	36	September, 1926	Soil, salt-grass sod.
6	24	September, 1926	Soil, salt-grass sod.
7	24	September, 1926	Sand, no vegetation.
8	48	September, 1927	Soil, salt-grass sod.
9	36	September, 1927	Soil, tules.

Fig. 7 shows the two water-surface evaporation pans; Fig. 8 shows one of the sand pans; and Figs. 8 and 9 also show the three small tubes placed in the soil pans for use in measuring depths to ground-water.

The water surface evaporation pans were designated Nos. 1 and 2. Pan No. 1, 4 ft. in diameter by 2 ft. deep, was set in the ground to a depth of 21 in., and was kept filled to within 3 in. of the rim. It was more nearly like a floating

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 852.

† *Loc. cit.*, p. 850.

pan than an ordinary ground pan, since the ground-water table was relatively close to the surface, the depth usually varying from about 8 to 20 in. Pan No. 2 was a standard U. S. Weather Bureau Class A evaporation pan, 4 ft. in diameter by 10 in. deep, mounted on a timber platform, having a surface about 7 in. above the ground.

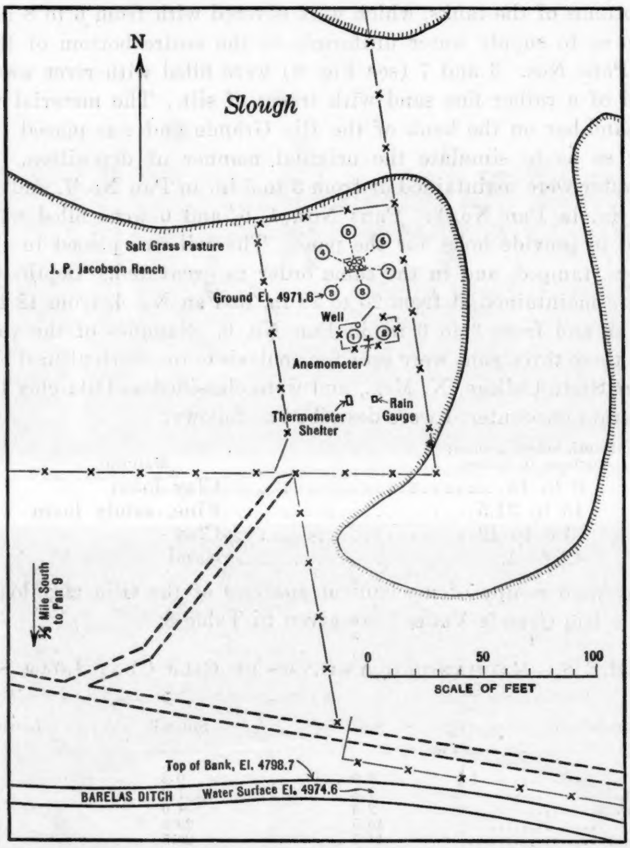

FIG. 6.—PLAN OF INSTALLATION AND TOPOGRAPHY AT LOS GRIEGOS EVAPORATION STATION.

The plans for the installation of the soil tanks were based on the author's Fort Collins station, but were modified somewhat so as to secure larger pans and tanks and more substantial pipe connections. Standard 18-gal. hot-water tanks, nominally 12 in. in diameter by 36 in. high, with the usual plumbing connections, were used as supply tanks; and standard farm stock tanks, approximately 4 ft. in diameter, were used as pans. All equipment was carefully tested for water-tightness before installation. The stock tanks were set in the ground, without the use of outside tank containers. The double-tank arrangement is necessary when the soil evaporation is determined by weighing, since then the tanks must be lifted out to be weighed. However,

when the Mariotte system is used, about the only advantage of the double tank is that the water-tightness of the outside container can be easily checked at any time by simply removing the inner tank and filling the outer one with water.

At the Los Griegos Station the water supply pipes were connected directly to the bottoms of the tanks, which were covered with from 6 to 8 in. of coarse gravel so as to supply water uniformly to the entire bottom of the test soil filling. Pans Nos. 3 and 7 (see Fig. 8) were filled with river wash material composed of a rather fine sand with traces of silt. The material was hauled from a sand-bar on the bank of the Rio Grande and was placed in the pans in water so as to simulate the original manner of deposition. Depths to ground-water were maintained at from 3 to 5 in. in Pan No. 7, and from about 20 to 28 in. in Pan No. 3. Pans Nos. 4, 5, and 6 were filled with the soil excavated to provide holes for the pans. The soil was placed in the pans in thin layers, tamped, and in the same order as excavated. Depths to ground-water were maintained at from 20 to 28 in. in Pan No. 4, from 12 to 19 in. in Pan No. 5, and from 3 to 6 in. in Pan No. 6. Samples of the various soils placed in these three pans were sent for analysis to the Agricultural Experiment Station at State College, N. Mex., and were classified as Gila clay loam. The various strata encountered were described as follows:

Depth below ground surface, in inches.	Material.
0 to 15	Clay loam
15 to 31.5	Fine, sandy loam
31.5 to 42	Clay
42 to 48	Sand

The average results of mechanical analyses of the Gila clay loam soils of the Middle Rio Grande Valley* are given in Table 8.

TABLE 8.—MECHANICAL ANALYSIS OF GILA CLAY LOAM SOILS.

Gradation.	Soil.	Subsoil.	Lower subsoil.
Fine gravel	0.0	0.3	0.3
Coarse sand	1.3	1.3	4.7
Medium sand	2.3	4.0	14.6
Fine sand	15.5	29.8	55.1
Very fine sand	14.7	26.5	17.6
Silt	35.3	17.2	3.7
Clay	30.9	16.2	4.1

A sample of the ground-water used in supplying both soil and water pans was taken August 28, 1926. It was analyzed at the Agricultural Experiment Station at State College, N. Mex., with the following results:

Chemical Analysis.	Mineral matter, in parts per 100 000.
Sodium carbonate	23.0
Sodium chloride	8.6
Sodium sulfate	13.2
Total	44.8

* As given in the *Soil Survey Bulletin*, 1912.

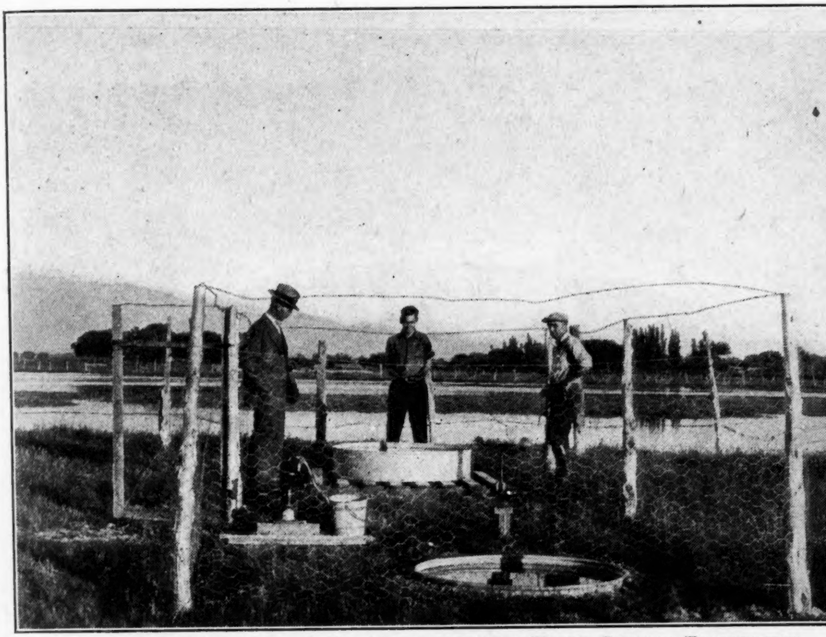

FIG. 7.—PANS NOS. 1 AND 2, FOR MEASURING FREE WATER SURFACE EVAPORATION, LOS GRIEGOS EVAPORATION STATION.

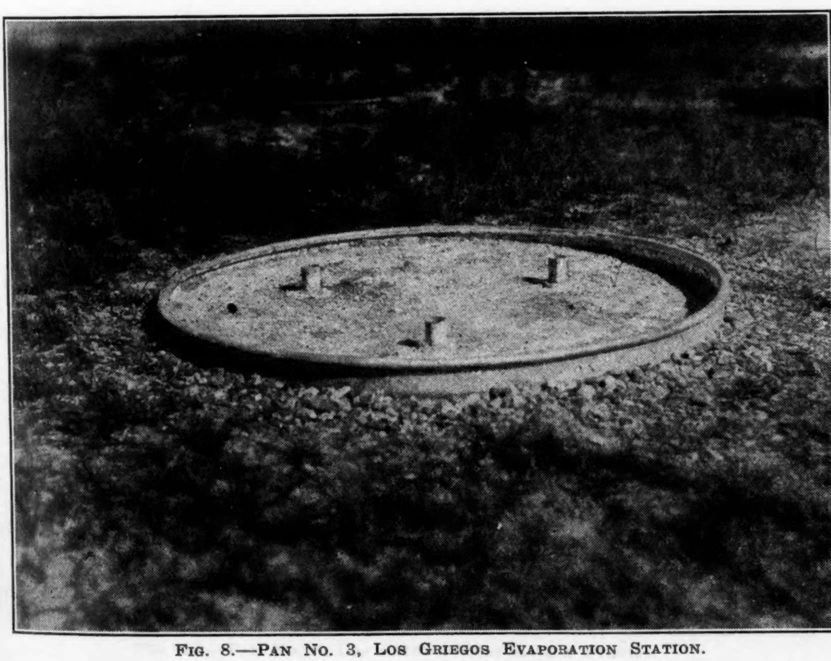

FIG. 8.—PAN NO. 3, LOS GRIEGOS EVAPORATION STATION.

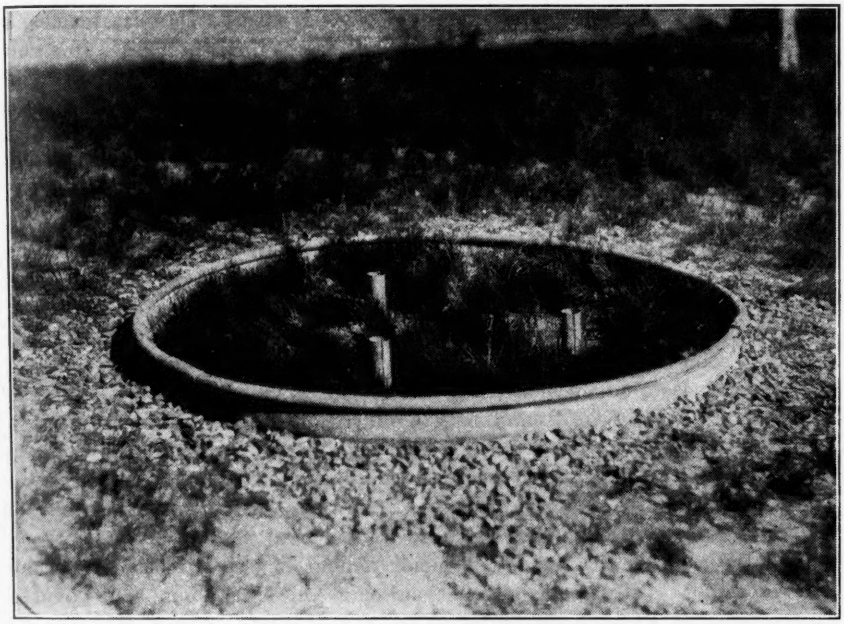

FIG. 9.—PAN NO. 4, LOS GRIEGOS EXPERIMENT STATION.

FIG. 10.—PAN NO. 6, LOS GRIEGOS EVAPORATION STATION.

The water in Pans Nos. 1 and 2 was renewed frequently, in order to avoid a concentration of alkaline salts which might influence the observed evaporation rate. Some difficulty with the soil tanks was anticipated because of the high content of sodium salts in the water used. However, the salt-grass grew vigorously, especially in Pan No. 6 which received a full supply of water.

During dry periods the evaporation rate at Pan No. 5 was decreased in comparison with that at the other pans, due to the formation of a fine alkaline dust on the surface of the soil. The evaporation rate at Pan No. 3 was also relatively low during such periods, due to the presence of a somewhat similar mulch made up of several inches of loose dry sand. These mulches greatly reduced capillary action.

For measuring depths to ground-water, three pipes were provided in each soil pan. The pipes terminated a short distance above the saturated layer of gravel at the bottom of the pan. However, the holes at the lower ends gradually opened until finally the elevations of the water-table in the pipes corresponded to the storage tank pressures. In the case of Pans Nos. 4 and 5 the depths to ground-water measured in the pipes were somewhat less than the actual depths to the water-table in the soil, as determined by additional holes, bored as needed, using care not to penetrate the gravel.

Violent fluctuations in elevation of ground-water surface at Pan No. 7 sometimes occurred due to sudden changes in meteorological conditions. A change of as much as 4 in. sometimes took place within a few hours for no apparent reason, probably due to unnoticeable changes in temperature and barometric pressure. The sudden occurrence of a strong, warm, dry wind would increase the rate of soil evaporation so much that the ground-water would be depleted more rapidly than it could be supplied by the Mariotte flask. Variations in ground-water level also occurred, due to variations in temperature and vacuum at the supply tanks. However, these fluctuations in ground-water level were relatively unimportant as regards the total monthly or annual evaporation losses.

In the fall of 1927 a study of the first year's records showed the need for a salt-grass sod pan in which the ground-water table could be kept at a somewhat greater depth than was possible with the pans already in use. Consequently, Pan No. 8 was installed, with provisions for maintaining the ground-water surface at a depth of approximately 3 ft. In filling Pan No. 8 the clay layer encountered between depths of 2 ft. 7½ in. and 3 ft. 6 in. was omitted so as to secure data more nearly comparable with those obtained at Pans Nos. 5 and 6. Another new pan (No. 9) was installed at that time. This was planted with tules and the ground-water kept at, or slightly above, the surface of the ground so as to determine the total depths evaporated and transpired in tule swamps. This pan was located in a small tule swamp in an old river channel about ½ mile south of the station. The swamp was usually dry in the fall of the year, for a period of from one to two months, due to the lowering of the ground-water level.

Tables 9 and 10* are summaries of the data secured during the first two years of operation. Table 9 gives the monthly meteorological data includ-

* "Preliminary Report on Middle Rio Grande Investigation, New Mexico," by E. B. Debler, M. Am. Soc. C. E., and C. C. Elder, Assoc. M. Am. Soc. C. E., U. S. Bureau of Reclamation, Denver, Colo., December 15, 1927.

TABLE 9.—METEOROLOGICAL RECORDS AT THE LOS GRIEGOS EVAPORATION STATION, SEPTEMBER, 1926, TO SEPTEMBER, 1928, INCLUSIVE.*

Year.	Month.	Mean temperature, in degrees Fahrenheit.	Total precipitation, in inches.	Mean† wind velocity, in miles per hour.	Mean relative humidity, percentage.	Depth to ground-water at Pan No. 1, in feet.	WATER SURFACE EVAPORATION. Pan No. 1, in inches.	Pan No. 2, in inches.	Ratio, Column (8) / Column (9) (in percentage).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1926	Sept...	67.2	1.04	2.7	59	1.2	5.58	7.02	79.5
1926	Oct....	57.3	1.03	2.4	64	1.4	3.77	5.00	75.4
1926	Nov....	44.2	trace	3.5	50	1.3	2.88	4.03	71.5
1926	Dec....	35.4	1.01	2.4	75	1.1	0.90¶	1.27¶	70.8
1927	Jan....	38.9	trace	2.0	64	1.1	1.01	1.45	69.7
1927	Feb....	44.0	0.34	3.6	56	1.2	2.35	3.23	72.8
1927	March.	46.5	0.50	4.7	50	1.2	4.61	6.23	74.0
1927	April..	53.8	0.17	4.0	45	0.9	5.71	8.37	68.2
1927	May...	61.8	trace	4.6	30	1.1	9.22	13.24	69.6
1927	June...	67.9	1.00	3.1	45	1.1	6.83	10.20	67.0
1927	July...	74.7	0.80	2.7	55	1.5	7.80	11.25	69.3
1927	Aug....	71.3	1.62	2.5	60	1.6	6.66	9.12	73.0
1927	Sept...	66.3	1.34	2.8	62	1.3	4.87	6.70	72.7
Averages and totals.......		55.2	7.81	3.19	54.7	1.23	56.61	80.09	70.7
1927	Oct....	54.7	0.19	2.2	48	1.3	3.86	5.54	69.7
1927	Nov...	46.6	0.04	2.5	55	0.9	2.56	3.38	75.7
1927	Dec....	31.5	0.06	3.2	68	1.0	1.07	1.45	73.8
1928	Jan....	33.5	0.00	2.6	33‡	1.1	1.08	2.02	53.5
1928	Feb....	38.2	0.32	3.6	44‡	1.1	1.90	2.90	65.5
1928	March.	47.1	0.06	3.6	31‡	1.1	4.31	6.17	69.9
1928	April...	52.0	0.75	5.1	29§	0.8	6.00	8.62	69.6
1928	May...	61.5	1.38	3.3	39§	0.8	5.34	8.12	65.8
1928	June...	68.5	0.00	3.8	22§	1.5	8.64	12.72	67.9
1928	July...	73.5	1.43	2.4	43§	2.3	7.40	11.00	67.3
1928	Aug....	70.2	2.65	2.6	62§	2.1	7.37	8.39	87.8
1928	Sept...	63.6	0.15	2.3	43‖	2.6	5.33	7.34	72.6
Averages and totals.......		53.4	7.03	3.10	43.1	1.38	54.86	77.65	70.7

* Station ½ mile east of Rio Grande and 4 miles northwest of Albuquerque, elevation 4 970 ft. above mean sea level.

† Anemometer 2 ft. above ground surface.

‡ Mean of 5:00 P. M. readings.

§ Mean of 5:00 P. M. readings for 24 days.

‖ Mean of 5:00 P. M. readings for 21 days.

¶ Pans covered with ice during most of December and up to January 9, 1927.

ing the measurements of free water-surface evaporation. It will be noticed that the ratios of monthly ground-pan evaporation (Column (8)) to monthly Class A pan evaporation (Column (9)) vary from about two-thirds to about three-fourths in most cases, and that the corresponding yearly ratios amount to 70.7% in each case (see Column (10)). Table 10 summarizes the data on evaporation and transpiration at the soil pans. Yearly totals and averages, for years ending September 30, are included in both tables, and ratios of soil evaporation to free water-surface evaporation at Pan No. 1 are given in Table 10. The yearly free water-surface evaporation averaged about 56 in. at Pan

TABLE 10.—EVAPORATION AND TRANSPIRATION LOSSES AT LOS GRIEGOS EVAPORATION STATION.

(All data in feet.)

Year.	Month.	Water surface evaporation Pan No. 1.	EVAPORATION FROM SAND AND RIVER WASH MATERIALS.				EVAPORATION AND TRANSPIRATION FROM SALT-GRASS SOD SURFACES.								Tule swamp, Pan No. 9, evaporation and transpiration.
			Pan No. 3.		Pan No. 7.		Pan No. 4.		Pan No. 5.		Pan No. 6.		Pan No. 8.		
			Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	
1926	Oct.	0.314	0.19	2.42	0.26	0.37	0.10	2.07	0.18	1.35	0.29	0.48	......	....	....
1926	Nov.	0.240	0.10	2.32	0.23	0.25	0.01	2.32	0.08	1.15	0.08	0.40	......	....	....
1926	Dec.	0.075	0.12	2.10	0.08	0.20	0.09	2.08	0.09	0.96	0.07	0.20	......	....	....
1927	Jan.	0.084	0.02	2.11	0.12	0.19	0.01	1.84	0.01	1.17	0.03	0.26	......	....	....
1927	Feb.	0.196	0.10	2.33	0.20	0.35	0.04	1.82	0.07	1.24	0.07	0.39	......	....	....
1927	March	0.384	0.17	2.36	0.40	0.28	0.05	1.67	0.11	1.10	0.14	0.40	......	....	....
1927	April	0.476	0.17	2.38	0.47	0.45	0.01	1.67	0.12	1.23	0.26	0.47	......	....	....
1927	May	0.768	0.16	2.49	0.68	0.41	0.05	2.30	0.26	1.22	0.59	0.37	......	....	....
1927	June	0.569	0.16	2.40	0.57	0.36	0.23	2.23	0.38	1.28	0.59	0.41	......	....	....
1927	July	0.650	0.11	2.50	0.63	0.39	0.29	2.25	0.51	1.27	0.75	0.41	......	....	....
1927	Aug.	0.555	0.20	2.43	0.54	0.34	0.35	2.28	0.61	1.12	0.67	0.40	......	....	....
1927	Sept.	0.406	0.14	2.24	0.40	0.34	0.28	2.30	0.35	1.06	0.49	0.39	......	....	....
Averages and totals...		4.717	1.64	2.34	4.58	0.33	1.51	2.07	2.77	1.18	4.03	0.38	......	....	....
Percentages...		100.0	34.8	....	97.1	....	32.0	....	58.8	....	85.4	....	......	....	....
1927	Oct.	0.322	0.12	2.38	0.32	0.20	0.16	2.16	0.27	1.34	0.29	0.47	0.04	3.06	0.23
1927	Nov.	0.213	0.08	1.86	0.18	0.42	0.18	2.02	0.10	1.16	0.09	0.45	0.02	3.08	0.15
1927	Dec.	0.089	0.13	1.59	0.11	0.19	0.05	1.89	0.05	1.18	0.05	0.39	0.02	3.10	0.08
1928	Jan.	0.090	0.03	2.10	0.13	0.33	0.01	2.17	0.01	1.10	0.03	0.39	0.01	3.07	0.10
1928	Feb.	0.158	0.08	2.22	0.17	0.23	0.04	2.17	0.03	1.19	0.05	0.40	0.03	3.04	0.14
1928	March	0.359	0.09	2.38	0.35	0.31	0.03	2.10	0.05	1.62	0.09	0.49	0.01	3.04	0.30
1928	April	0.500	0.10	2.17	0.43	0.31	0.10	2.10	0.16	1.30	0.17	0.40	0.07	3.09	0.43
1928	May	0.445	0.16	1.78	0.44	0.28	0.22	2.09	0.32	1.30	0.40	0.42	0.13	3.09	0.44
1928	June	0.720	0.06	2.18	0.58	0.42	0.18	2.17	0.45	1.41	0.74	0.53	0.02	3.09	0.89
1928	July	0.617	0.15	2.26	0.57	0.34	0.32	2.33	0.60	1.43	0.84	0.51	0.16	3.09	1.09
1928	Aug.	0.614	0.20	1.90	0.47	0.25	0.41	2.27	0.56	1.41	0.64	0.48	0.28	3.04	0.89
1928	Sept.	0.444	0.33	2.02	0.37	0.25	0.19	2.25	0.33	1.41	0.48	0.51	0.05	3.07	0.65
Averages and totals...		4.571	1.53	2.07	4.12	0.29	1.89	2.14	2.93	1.32	3.87	0.45	0.84	3.07	5.39
Percentages...		100.0	33.5	....	90.2	....	41.3	....	64.1	....	84.7	....	18.4	....	117.9

* Average depth of water-table below ground surface.

No. 1, and about 79 in. at Pan No. 2. Annual losses at the salt-grass sod pans varied from 0.84 ft. at Pan No. 8, where the depth to ground-water averaged 3.07 ft., to 4.03 ft. at Pan No. 6 where the average depth to ground-water was about 3 in. Average depths to ground-water at Pans Nos. 4 and 5, as given in Table 10, may be a few inches too low for the reason previously discussed. The yearly loss at Pan No. 9, which was planted with tules, was 5.39 ft.

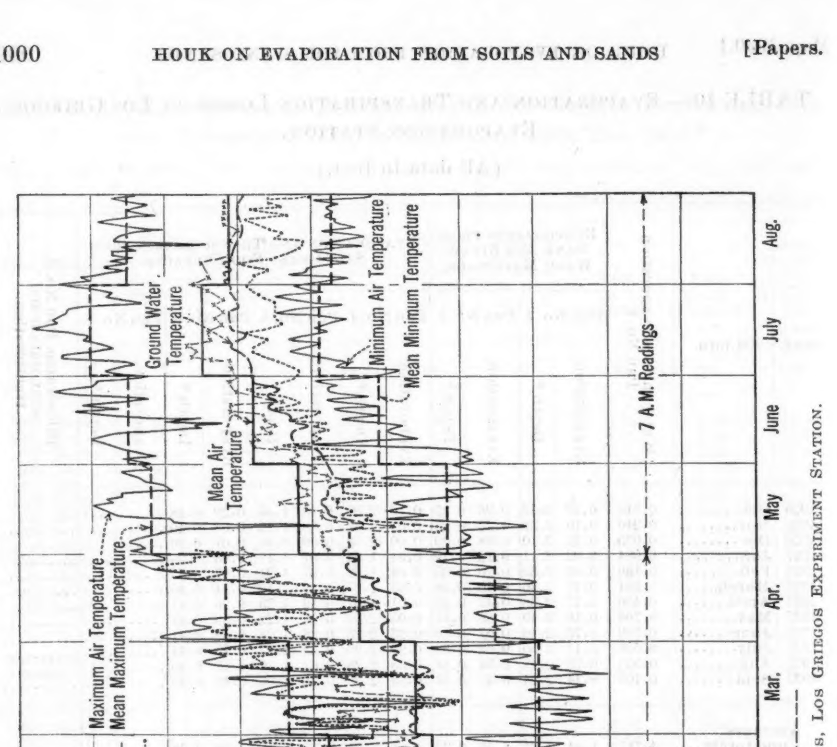

FIG. 11.—AIR AND WATER TEMPERATURES, LOS GRIEGOS EXPERIMENT STATION.

Fig. 11 shows the daily meteorological records, including temperatures of the ground-water and of water in Pans Nos. 1 and 2, during the year ending August 31, 1927. Daily readings were taken at about 7:00 A. M. during the months of May, June, July, and August, and at about 5:00 P. M. during the remainder of the year. The ground-water temperatures did not vary greatly from the mean monthly air temperatures during any part of the year, although they were slightly lower than the air temperatures during the summer when the measurements were made in the morning, and slightly higher than the air temperatures during the remainder of the year when the observations were made late in the afternoon. Water temperatures in the evaporation pans were nearly as great as the maximum air temperatures when the observations were made at 5:00 P. M. and slightly lower than the mean air temperatures when the readings were made at 7:00 A. M. Water temperatures in Pan No. 1 were a few degrees lower than those in Pan No. 2 as would be expected.

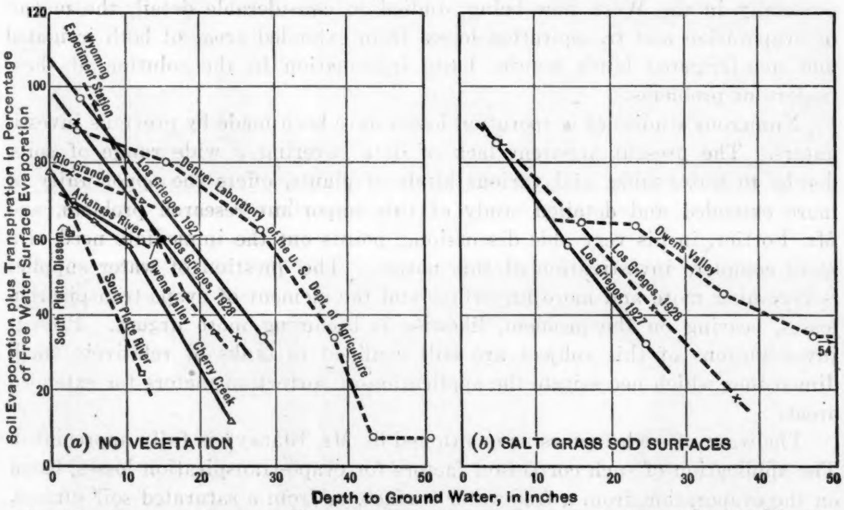

FIG. 12.—RELATION BETWEEN TOTAL ANNUAL EVAPORATION AND DEPTH TO GROUND-WATER.

Fig. 12(*a*) shows the annual soil evaporation at Pans Nos. 3 and 7, in percentage of free water-surface evaporation, plotted against the average depths to ground-water. For comparative purposes similar data obtained at the Wyoming Agricultural Experiment Station,* at the Denver Laboratory of the U. S. Department of Agriculture,† and in Owen's Valley, California,‡ have been added on the diagram. The data presented by the author have not been added since they are similarly shown in Fig. 4.§ Fig. 12 (*a*) would indicate that but little, if any, evaporation occurs from the surfaces of bare, sandy soils when the depth to ground-water exceeds about 4 ft.

* *Bulletin 52*, Wyoming Agricultural Experiment Station.

† "Evaporation from the Surfaces of Water and River Bed Materials," by R. B. Sleight, *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

‡ "An Intensive Study of the Water Resources of a Part of Owen's Valley, California," by Charles H. Lee, U. S. Geological Survey, *Water Supply Paper 294*.

§ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 854.

Fig. 12 (*b*) shows the annual soil evaporation and transpiration at the salt-grass sod tanks, Pans Nos. 4, 5, and 6, the Owen's Valley data being added for comparison. In this case the results of the Owen's Valley investigations constitute the only similar data available. It will be noticed that the ratios of soil evaporation and transpiration to free water-surface evaporation were appreciably lower at the Los Griegos Station than at the Owen's Valley Station. The Los Griegos records were also more consistent as regards relation of evaporation losses to average depths to ground-water. The Los Griegos experiments seem to indicate that the loss of soil moisture by evaporation and transpiration, from the Gila clay loam covered with salt-grass sod, practically ceases when the depth to ground-water reaches about 4 ft.

RALPH L. PARSHALL,* ASSOC. M. AM. SOC. C. E. (by letter).†—The writer is gratified in the general interest shown in the subject of evaporation from moist soils, by the several discussions. Because of water supply problems, especially in the West, now being studied in considerable detail, the matter of evaporation and transpiration losses from extended areas of both irrigated and non-irrigated lands require basic information in the solution of these important problems.

Numerous studies of evaporation losses have been made by previous investigators. The present apparent lack of data covering a wide range of soils, depths to water-table, and various kinds of plants, offers the opportunity of more extended and detailed study of this important research problem, and Mr. Fortier, in his very able discussion,‡ points out the immediate need of a more complete investigation of this nature. The question of water supplies is becoming more and more important and the element of evapo-transpiration losses, bearing on the problem, likewise is becoming more urgent. Present investigations of this subject are still confined to tanks of relatively small dimensions which necessitate the application of correction factors for extended areas.

The want of such factors as mentioned by Mr. Blaney§ is fully appreciated. The application of such correction factors for evapo-transpiration losses, based on the evaporation from a free water surface, or from a saturated soil surface, can not be reasonably approximated. The vegetative draft on the soil moisture is much more than is commonly supposed and, therefore, until more extensive studies are made on a large scale, covering a rather wide range of conditions, the engineer will be obliged to apply very uncertain corrections in summing up the losses due to evaporation and plant use. The present investigation by Messrs. Blaney and Young in California‖ is without doubt the most extensive and detailed study yet attempted, and from this work will come a substantial advancement in the knowledge of this important question.

Mr. Sonderegger's statement‖ that the losses from the fine river-bed sands were disturbed, is true. However, as these sands were originally laid

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* Senior Irrig. Engr., Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, Fort Collins, Colo.

† Received by the Secretary, March 11, 1930.

‡ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1881.

§ *Loc. cit.*, p. 1886.

‖ *Loc. cit.*, p. 1885.

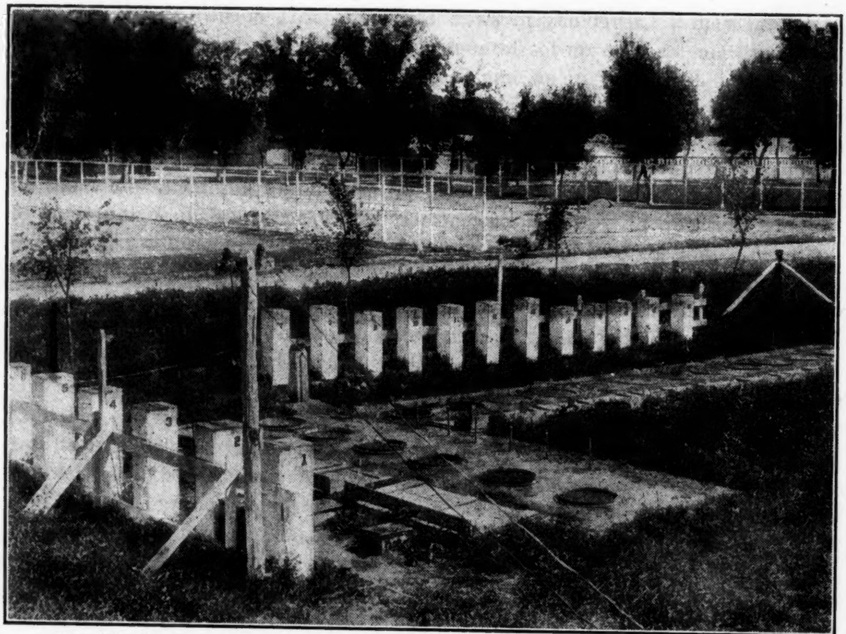

FIG. 13.—GENERAL VIEW OF ARRANGEMENT OF SOIL TANKS AND METEOROLOGICAL EQUIPMENT.

FIG. 14.—HOUSING FOR MARIOTTE TUBE REGULATION.

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down by the deposition from the flood flows of the river, it may be fairly concluded that, in the case of the experimental set-up of these materials, the density or compaction was essentially the same as in the original bed. The practical engineering use of the data resulting from experimental studies must of necessity be applied to a very wide range of soil conditions. To assume that the results of an investigation of a single soil condition are applicable to the wide variation found over an extended area, would therefore seem to be rather doubtful. The writer is somewhat inclined to question whether any marked difference in the rate of evaporation from a cored sample or otherwise would be evident in an experimental set-up.

The use of sprinkling in the application of water to the soil, is believed to be uneconomical from two standpoints: First, if the application is moderate, the cooling of the ground surface and the temporary linking up of the capillary movement have been shown to deplete the soil moisture; and, second, the fact that the spray vastly increases the evaporating area of the water surface results in a much greater loss than if the water was applied as a stream in contact with the ground. As pointed out by Mr. Sonderegger, the classification of soil by the moisture equivalent percentage, from the standpoint of evaporating medium, is no doubt much more satisfactory than the simple classification of a mechanical analysis. It seems reasonable to suppose that possibly the rate of capillary movement in a soil column might be a better index than either the mechanical or the moisture equivalent. The apparent evaporation loss from soil surfaces where the water-table is at some depth, is, in effect, a function of the capillary movement of the moisture to the surface where it is dissipated as evaporation. The loss can only be as rapid as the capillary movement of the soil moisture from the water-table to the surface.

The proposal to place covers over the experimental soil tanks, as a means of protection against rain, may be considered questionable. Experience indicates that where rain falls on the exposed area, an element of uncertainty in the resulting records becomes apparent, because of the inability to determine the amount of the correction to be applied due to the precipitation. Splash from a dashing shower, wide rims on the tanks, and the inherent inaccuracy of measuring the rainfall in a standard rain gauge, all result in a doubtful correction factor; also, the soil surfaces, when disturbed by a wetting, immediately assume a different evaporating medium. To eliminate these uncertainties, covers were provided. The rate of evaporation with the covers on resulted in a marked decrease in the rate of loss. Ordinarily, these covers were on only for a short period to protect against a passing shower. Inasmuch as all tanks were covered for the same period, the relative losses may be consistently compared.

The discussion by Mr. Cummings* brings forward the interesting relation of the atmospheric component of energy to the phenomena of evaporation. Energy is expended in the dissipation of water by evaporation and when the surface of the evaporating medium is shaded, the incoming solar energy is reduced. The experiments in question were not in sufficient detail to draw

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2219.

conclusions as to this effect. The covers were partly opened at the ends and at one side, thus permitting some circulation of air currents. However, as the sun was shaded by clouds at the time the covers were in place, no direct relation of shaded and unshaded effects are possible. It is assumed that the most important factors tending to reduce the rate of evaporation when the covers are in place are the increased humidity due to the rain and the protection against air movement.

Mr. Houk's discussion* presents valuable data, especially for depths of soils exceeding those reported by the writer. At the time of the set-up of the apparatus to study the evaporation loss from moist soil surfaces, it was the intention to determine the relative losses for shallow depths of water-table for various types of fine sands and other soils. In 1913 a series of experiments were conducted by Mr. V. M. Cone to determine the rate of capillary rise in soil columns about 1 in. in diameter. It was found that, for a fine sandy loam soil, taken at Eads, Colo., more than 100 days were required for the moisture to reach a height of 58 in. For the river-bed sand, as used by Mr. Houk, the writer would be inclined to assume that the loss would approach zero for a water-table much less than 4 ft. For the medium fine sand (Fig. 4†) it seems evident that no loss would occur at about 20 in.

The data presented in Table 3‡ give the average daily loss per 24 hours for the period indicated. The first tabulation of the results of this investigation gave the average loss per 24 hours for each week and, in compiling these data, corrections were made where the covers were in place for several hours. For the most part the covers were put on just long enough to protect the tanks during a shower and for these short periods no attempt was made to apply a correction.

Fig. 13 is the general arrangement of soil tanks with housing over the Mariotte tube regulators. (See, also, Fig. 2.§) Fig. 14 shows Tank No. 13 with the exposed glass tube and adjacent fixed meter stick used in determining the evaporation loss. Previous to the use of the Mariotte principle as applied to maintaining and regulating the water supply for an evaporation tank, an arrangement very similar to the apparatus described by the writer was used by O. V. P. Stout, M. Am. Soc. C. E., and Carl Rohwer, Assoc. M. Am. Soc. C. E., in the study of seepage losses from canals in California.

* See p. 989.

† *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 854.

‡ *Loc. cit.*, p. 850.

§ *Loc. cit.*, p. 845.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS

Discussion*

By A. L. Sonderegger, M. Am. Soc. C. E.†

A. L. Sonderegger,‡ M. Am. Soc. C. E. (by letter).§—The extensive discussion of this paper is proof of the interest which the subject has aroused of late years in the semi-arid West. It has been fruitful of many new and interesting ideas and facts and not a few contradictory statements; also, some criticism. A discussion of natural phenomena, such as rainfall penetration and run-off, and particularly of the methods for their determination which the paper was intended to bring out, must, of necessity, make reference to specific cases.

Mr. Houk,‖ in the closing paragraph of his discussion, summarizes the general conclusion reached, namely, that the geological, topographical, and meteorological conditions which affect rainfall, run-off, and percolation, are so many, so different, and so variable that general conclusions, or the establishment of general laws, cannot be based upon studies of an individual area or a group of areas. The discussion has brought out that present knowledge of the many factors which affect the disposal of rainfall is only fragmentary. There is little or no information available as to the consumptive use of water by native vegetation except as expressed in the general rainfall-run-off relation. The latter indicates that, generally speaking, from 40 to 100% of the rainfall is consumed by the cover. It goes without further explanation that the conservation of rainfall through the control of the consumptive use in the water-sheds would be of unestimable value.

To further such results, it is imperative that country-wide, systematic investigations be made of the consumptive use of the water-shed cover, including the water needs of individual plant species, the distribution of rainfall,

* Discussion of the paper by A. L. Sonderegger, M. Am. Soc. C. E., continued from April, 1930, *Proceedings*.

† Author's closure.

‡ Cons. Engr., Los Angeles, Calif.

§ Received by the Secretary, March 21, 1930.

‖ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 132.

the effect of over-year storage, and the multiplicity of other factors which affect penetration and run-off.

The idea is repeatedly advanced by discussers that rainfall, after it reaches the ground, remains evenly distributed, practically, and that absorption is uniform to the depth of plant roots, regardless of the ever-present irregularities of the surface of the ground and the lack of uniformity of the residual soils of the mountain water-sheds or of alluvial deposits in the foothill areas and steep valleys adjacent to mountain ranges. Hence, the discussers repeatedly asserted that no deep penetration can occur unless the field capacity of the soil down to the root limit is first satisfied over the entire area under consideration.

Doubt has been expressed by some of the discussers as to the comparatively large quantities of deep penetration in the valley floor which the writer believes become available. In this connection, the study of water supply from closed basins should furnish conclusive evidence. Such studies are available for the San Bernardino Basin in the water-shed of the Santa Ana River, above Prado, in Southern California. Reference is here made to two independent investigations of the water supply of this basin; one by the Engineering Offices of J. B. Lippincott, M. Am. Soc. C. E., in Los Angeles, Calif.,* and one by the State Engineer's Office of California.†

The water-shed of the Santa Ana River above Prado is a closed basin; it is bordered on all sides by mountain ranges embracing a valley area, which is a sunken basin many hundred feet in depth, filled with alluvial materials. This area is capable of absorbing a considerable percentage of water. A large portion of the tributary mountain water-shed enjoys a prolific rainfall and run-off. The latter has been measured by the U. S. Geological Survey in the Santa Ana River and numerous creeks, for a sufficient period of time to permit of accurate estimates of total run-off reaching the valley. The principal stream is the Santa Ana River (water-shed, 203 sq. miles). The river, after traversing the basin and intercepting its tributaries, enters the gorge of Lower Santa Ana Canyon where a gauging station has been maintained near Prado since 1919-20. There is also available a large number of current-meter measurements at this point, made at more or less frequent intervals, since the early Nineties. The cross-sectional area of the underflow at this point is known. Surveys as to the use of water for domestic purposes and irrigation within the basin are available; surveys also are available of the moist areas from which water evaporates. Furthermore, there are rainfall records, covering long periods, of a number of stations in the valley floor and fluctuations of the water level in wells. In short, there are available comparatively complete data to establish the water supply, use, and waste in this closed basin.

In the Lippincott report previously mentioned, the possible sources of water supply, mountain water-sheds, foothill areas, and valley floor, were

* Report on Water Conservation and Flood Control on the Santa Ana River for Orange County, July, 1925, Table 16.

† "Santa Ana River Investigation," *Bulletin No. 19*, State of California, Dept. of Public Works, Div. of Eng. and Irrig. (as yet not published).

credited with the quantities listed in Table 19. These data are the averages for the 29-year period from 1895-96 to 1923-24.

The water crop attributed to the valley floor, 580 sq. miles in area, is nearly all deep penetration and amounts to 114 000 acre-ft. per annum, or more than 30% of that of the entire water-shed.

TABLE 19.—TOTAL SEASONAL WATER CROP OF SANTA ANA RIVER ABOVE PRADO, CALIF.

Region.	Area, in square miles.	WATER CROP, 29-YEAR MEAN.	
		Total, in acre-feet.	Acre-feet per square mile.
Mountain water-shed, north and east	547.01	193 142	353.08
Temescal Creek	200.74	1 509	7.52
Foothill areas, north and northeast	68.26	22 328	327.10
Foothill areas, northwest	48.16	27 168	564.12
Foothill areas, south	40.73	2 748	67.46
Valley floor, north and northeast	245.88	46 644	189.70
Valley floor, Chino Creek	249.52	54 024	216.51
Valley floor, south	84.89	13 428	158.18
Total	1 485.19	360 991	243.
Santa Ana River, at Prado	....	153 102	....

For the purpose of comparison with Table 19, estimates of rainfall penetration in the valley floor of San Bernardino for the seasons 1926-27 and 1927-28 are presented in Tables 20 and 21.* These statements are based upon penetration experiments and studies by Mr. Blaney.†

The water supply characteristics of the two seasons are expressed in the index of wetness (percentage of long-period mean rainfall) and index of run-off (percentage of long-period mean run-off of the mountain streams), to wit:

	1926-27.	1927-28.
Index of wetness (City of San Bernardino)....	128	87
Run-off index	147	34

Tables 20 and 21 indicate that under certain conditions the water supply from rainfall penetration in the valley may assume substantial proportions and may become a factor of economic importance.

Mr. Blaney has contributed a valuable discussion and much useful data on evaporation and transpiration losses obtained in Southern California by the method of systematic soil moisture determinations.† For areas with various types of soil, cover, and culture, this in many cases may be the only method available for the determination of the percentage of deep penetration. As stated in the paper,‡ the method is subject to the errors resulting from

* *Bulletin No. 19*, State Dept. of Eng., California, Tables 22-*A* and 22-*B*.

† *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2223.

‡ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1144.

the assumption that rainfall remains uniformly distributed after it reaches the ground and that there is no erratic variation of soil texture. The method is not so successful with very gravelly soils, because it is difficult to obtain samples except by digging shafts.

TABLE 20.—SAN BERNARDINO BASIN: SUMMARY OF ESTIMATES OF PENETRATION WITHIN VALLEY FLOOR FOR 1926-27 AND 1927-28.

Basin.	Area, in square miles.	Mean rainfall, in inches.	MEAN PENETRATION.		Total penetration, in acre-feet.
			Inches.	Acre-feet per square mile.	
Upper:					
1926-27	124.1	21.9	7.62	407	50 700
1927-28	124.1	17.6	1.68	86	10 700
Jurupa:					
1926-27	99.1	18.4	4.17	223	22 000
1927-28	99.1	16.1	.77	41	4 100
Cucamonga:					
1926-27	268.9	22.9	8.85	472	126 700
1927-28	268.9	15.2	2.93	156	41 700
Temescal:					
1926-27	55.9	16.0	3.02	161	9 000
1927-28	55.9	14.0	1.52	82	4 600
Lower (Coastal Plain):					
1926-27	305.6	16.1	4.87	260	79 200
1927-28	305.6	15.3	4.70	251	76 900

The extensive and thorough investigations now carried on by the Division of Agricultural Engineering, U. S. Department of Agriculture, some of which are under the immediate direction of Mr. Blaney, will throw light on many phases concerning the problem of rainfall penetration. Comparison between

TABLE 21.—SAN BERNARDINO BASIN: ESTIMATE OF MEAN PENETRATION VALUES WITHIN VALLEY FLOOR, IN ACRE-FEET PER SQUARE MILE, FOR VARYING MEAN RAINFALL.

Mean rainfall, in inches.	MEAN PENETRATION, IN ACRE-FEET PER SQUARE MILE.				
	Upper Basin.	Jurupa Basin.	Cucamonga Basin.	Temescal Basin.	Lower Basin (Coastal Plain)
9	0	0	0	0	0
12	0	0	27	5	91
15	0	0	150	123	224
18	118	187	272	250	360
21	336	336	390	390	490
24	500	470	520	520	620

penetration resulting from protracted storms and sprinkling experiments carried on in dry weather should lead to conclusions regarding the effects of atmospheric conditions, which are difficult of analysis.

Mr. Sopp's statement* as to the results of soil sampling in a shaft 170 ft. deep is in confirmation of the writer's views and experience, and contradicts the theory advanced by many engineers that rainfall of the intensity charac-

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2227.

teristic of Southern California would not, and could not, penetrate a soil prism 170 ft. What most water supply engineers did not expect, however, and some still dispute, is that the percentage of rainfall reaching the water-table is as large as the writer's investigations have led him to conclude.

Mr. Troxell* takes issue with the analysis of mountain run-off as shown in Fig. 8† and with the determination of the consumptive use of the cover from the "total run-off" curve. The values obtained by the writer's method represent averages over a period of years in which the effects of individual years are ironed out. For larger water-sheds with perennial flow, this method is probably the only one that is practicable, since it becomes difficult to trace the effect of individual years on account of the long-period storage which takes place. For a small water-shed like that of Devil Canyon, the following considerations will apply:

(a) For a dry year, preceded by a wet year, the curve gives values of consumptive use that are too large because of a draft on over-year storage (see year 1923, on Fig. 8);
(b) For a wet year, preceded by a dry year, the curve gives values of consumptive use that are too low, because a part of the rainfall remains in storage (see year 1922, on Fig. 8); and
(c) For the second and third years of a series of years of the same wetness, the curve gives correct values (see years 1920 and 1921 and years 1924 and 1925).

The larger the water-shed the more complex becomes the effect of hold-over storage. Considering further that, as a rule, data relative to the distribution of rainfall over large water-sheds are fragmentary, that no two seasonal rains fall in the same manner, and that moisture conditions and conditions of the cover of a water-shed are likely to vary from year to year, practical results can be obtained only by methods which iron out the irregularities of individual years.

Just why Mr. Troxell criticizes the storm run-off curve of Fig. 8 is not clear. The curve speaks for itself; it deviates very little from the construction points. The discusser states‡ that "storm run-off depends not on seasonal rainfall, or even storm periods, but on the instantaneous rate of rainfall and the condition of the canyon storage". A determination of the storm run-off of a mountain water-shed from the instantaneous rate of rainfall is too impractical to deserve serious consideration. The method is used to estimate peak flows in urban areas, where the instantaneous rate of rainfall is known. In mountain water-sheds rainfall stations, as a rule, are few and far between, and daily or weekly rainfall records are the best to be expected.

Mr. Troxell's application of the run-off curves of Strawberry and Waterman Canyons to Lone Pine Creek§ is hardly fair. The first two water-sheds are short, steep slopes on the south side of San Bernardino Mountains, exposed directly to the moisture-laden clouds drifting in from the ocean and receiving a very prolific rainfall. Lone Pine Creek, on the other hand, is on the lee or

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1935.
† *Loc. cit.*, May, 1929, Papers and Discussions, p. 1156.
‡ *Loc. cit.*, September, 1929, Papers and Discussions, p. 1936.
§ *Loc. cit.*, p. 1938.

desert side of the highest peak of the Sierra Madre; its northerly slopes are formed by a low range of desert hills. The creek bed runs along the San Andreas fault rift for a distance of 10 miles; climatic conditions are those of a desert valley. No wonder the results of Mr. Troxell's comparison were disappointing.

Mr. Rowe's discussion* divides itself into two distinct sections. The first refers to the consumptive use of water-sheds near San Bernardino, with which Mr. Rowe is very familiar. His data as to the effect of a denudation of the brush cover by fire are very instructive. The quantity of deep penetration from rainfall of 250 acre ft. per sq. mile as determined by the writer for the San Bernardino area, is questioned by Mr. Rowe.

The second section of Mr. Rowe's discussion refers to the determination of rainfall penetration in Pauba Valley and on the Big Mesa of Temecula River. A casual trip over the Big Mesa is hardly enough to enable a person to become familiar with this water-shed or with the writer's investigations.

Relative to the penetration experiments on the Big Mesa,† Mr. Rowe refers to the existence of hardpan at 4 ft. below the surface and states that this is the greatest depth to which rainfall ever penetrated. The writer made borings on the Big Mesa after the great storm of February, 1927. Some of the samples taken from approximately 16-ft. depths were wet and plastic. This moisture must have found some way to get below the so-called impervious layer of hardpan.

Mr. Rowe attributes special significance to the fact that on the Big Mesa there are two wells in which water stands higher by more than 40 ft. and 65 ft., respectively, than in adjoining wells. In reply, it may be stated that it is conceivable that a well would terminate in a thick clay stratum which is above the general water-table. Such a well, naturally, would drain the suspended water-table produced by the clay stratum in accordance with Fig. 1.‡ The well, therefore, would hold water, but its level would be above that of the general water-table.

Engineers interested in rainfall penetration are indebted to Mr. Lee§ for an instructive discussion which has brought to light many new and important points. The discussion relative to mountain run-off and "evaporation opportunity" is particularly useful.‖

Referring to the discussion of the computation of rainfall penetration from test wells in Pauba Valley,¶ Mr. Lee's method of determining the factor to convert the observed rise of a water-table to the equivalent depth of water, is a decided improvement over the assumption of a flat percentage; but the elements which enter into such computations are more in detail than would ordinarily be available to the engineer in making a water supply study, except for small areas. The writer's percentage of porosity of 25% was the result of the study of a number of well logs.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2695.
† *Loc. cit.*, p. 2702.
‡ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1141.
§ *Loc. cit.*, April, 1930, Papers and Discussions, p. 787.
‖ *Loc. cit.*, p. 791.
¶ *Loc. cit.*, p. 798.

The following remarks relative to the Pauba Valley test wells are applicable to the discussions by both Messrs. Lee and Rowe.

Approximately sixty-four wells were used in making the study of rainfall penetration in the Pauba Valley. Practically all these wells were shallow; many of them penetrated only 2 or 3 ft. into the water-plane. Most of the wells were located in, or adjacent to, fields which were irrigated approximately once a month during the summer season. These wells within the irrigated area showed a rise following the application of irrigation water very similar to the rise which followed rain storms. The deduction was made that if an application of irrigation water of 5 to 6 in., at a time when no storm run-off was present in the river, produced a rise in the wells, a similar rise would occur after a rainfall of 5 or 6-in. intensity to, roughly, the same amount, and it would be logical to conclude that this rise had been produced by the rainfall rather than by transmitted pressure.

Some of these irrigation rises, from May to September, are shown in Fig. 5,* but they are much clearer in corresponding graphs of many of the other wells. It was impossible to present all the data upon which the deductions were based.

The conclusion was advanced by the discussers that transmission of pressure, rather than local rainfall, is responsible for the rise in the test wells. This is a question which may be of general interest. In explanation of any such theory, it should be stated that Pauba Valley is a closed basin; the river enters by way of Nigger Canyon Gorge and leaves by way of Temecula Gorge,† with bed-rock outcropping in the river bed in both gorges. The gradient of the river averages about 38 ft. per mile. Below the upper gorge the river spreads over a wide outwash area, or débris cone, covered by deposits of sand and gravel. It is in this region that effective absorption occurs, because the entire summer flow, as a rule, seeps away. Pauba Valley is several hundred feet in depth and is filled with alluvial materials capable of absorbing about 25% of water as disclosed by the well logs. As the underflow approaches the lower gorge it is obstructed by the bed-rock formation and is forced to the surface, producing a rising stream which begins as a trickle about 6 miles above the lower gorge and increases to about 10 sec-ft. as the river approaches the gorge. The combined effect of the bed-rock and the resistance to percolation by the valley fill produces a pressure in the ground-water strata, responsible for the rising stream and for the flow in artesian wells. This is a phenomenon quite common with Southern California streams.

The water-table of the outwash area presents the fountain-head of this pressure. The process of ground-water movement, however, is believed to be a combination of pressure and percolation, the latter being the predominant factor for the reason that this basin, although only 9 miles in length between gorges, has the effect of long-period regulation of stream flow, with a summer flow varying from 8.5 to 12.5 sec-ft. In 1900, after a period of seven predominantly dry years, there was still a perennial flow at the lower gorge of about 10 sec-ft. during the summer months.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1149.

† *Loc. cit.*, p. 1147.

If the rise in the test wells was due to transmission of pressure, it would have had to occur either simultaneously with, or after, the appearance of the flood flow in the stream. As a matter of fact, a flood would naturally lag behind the downpour. It should also be noted that the flood channel in the outwash area may occupy only a fraction of the total width of the latter, unless the flood were to assume extraordinary proportions; that under ordinary flood conditions a certain lateral percolation must take place in the outwash area and a rise of the water-table over a considerable region before a transmission of pressure would affect the entire valley below. The lateral percolation observed, occurred under an oblique angle to the axis of the stream bed, and at rates varying from 16 to 33 ft. per day.

On certain of the test wells continuous records were kept which indicated that the rise preceded the flood flow at the outwash area. Many of the graphs also show that the test wells reached a peak before the wells in the outwash area above them and which, of necessity, would be the origin of any transmitted pressure. In most cases, after the occurrence of these peaks the test wells declined rapidly, while the wells in the outwash area were still approaching their high point. It would be logical to conclude that if the fluctuations in the test wells were dependent upon transmitted pressure, both the rise and the decline would follow the corresponding rise and decline of the levels in the wells in the outwash area rather than precede it.

Any conclusion that the rise in a test well might be due to a rise in the river opposite that well cannot be reached because the elevation of the water-table was higher than the river. It also appears that both Mr. Lee and Mr. Rowe expect a uniform manifestation of rainfall percolation, regardless of the unevenness of the ground and lack of uniformity of soil formation. Collection of rain water in depressions may readily account for a rise in some test well, greater than the rainfall would warrant.

TABLE 22.—Pumping Test at Well No. 131.

Date.	Time.	Depth to water, in second-feet.	Output, in second-feet.	Remarks.
May 18, 1928	11:00 A. M.	63.04	......	......
May 23, 1928	10:00 A. M.	62.70	......	......
May 23, 1928	11:00 A. M.	82.40	0.341	Pumping at intervals; many interruptions.
May 24, 1928	7:00 A. M.	83.77	0.351	
May 25, 1928	7:00 A. M.	85.65	0.382	
		Changing casing		
May 25, 1928	9:00 P. M.	111.58	0.789	......
May 29, 1928	12:00 M.	105.10	0.837	Constant pumping for 29 hours 40 min.
May 30, 1928	5:40 P. M.	115.70	0.799	Stopped pumping
May 30, 1928	5:48 P. M.	78.69	......	Not pumping.
May 30, 1928	6:40 P. M.	71.63	......	Not pumping.
May 30, 1928	7:40 P. M.	70.49	......	Not pumping.
May 31, 1928	7:00 A. M.	67.66	......	Not pumping.
June 1, 1928	7:10 A. M.	66.10	......	Not pumping.
June 10, 1928	9:35 A. M.	63.70	......	Not pumping.
June 15, 1928	4:00 P. M.	63.41	......	Not pumping.

Relative to the matter of rainfall penetration on the Big Mesa, which is doubted by Mr. Lee and Mr. Rowe, the question resolves itself ultimately

as to the existence of a general water-table beneath the Mesa. A pumping test made from May 18 to June 15, 1928, on Well No. 131, located on the Mesa about 1½ miles east of Temecula (after the paper had been written), tends to support the writer's conclusions. The well is 12 in. in diameter and 604 ft. in depth. Its log shows 61 ft. of aquifer, 18 ft. of undetermined water-bearing capacity, and 525 ft. of impervious strata. The general features of the test are shown in Table 22.

The total quantity of water pumped was 3.63 acre-ft. The volume of water pumped and the rate of recovery of the water-shed would seem to warrant the conclusion that a general water-table underlies the Big Mesa.

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PAPERS AND DISCUSSIONS

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REGULATION OF LEVELS, FLOW, AND NAVIGATION, NIAGARA RIVER:

SUMMARY AND CONCLUSIONS OF VARIOUS STUDIES

Discussion*

By G. B. PILLSBURY, M. AM. SOC. C. E.†

G. B. PILLSBURY,‡ M. AM. SOC. C. E. (by letter).§—This paper was prepared to introduce and invite discussion on lake regulation as affected by the Niagara River. In addition to the discussions presented, attention is invited the able treatment of the regulation of Lake Erie with reference to power, contained in the paper by Norman R. Gibson, M. Am. Soc. C. E., entitled "Niagara Power"‖ and in discussion¶ thereon by William Kelly, M. Am. Soc. C. E.

To the Engineering Profession at large, as well as to the general public, the feasibility and desirability of the regulation of the lakes appear axiomatic. The writer must confess that he approached the subject, some years ago, in a more than hopeful frame of mind. It is difficult to appreciate the extent to which widespread interests have adjusted themselves to the present regimen of the lakes and their outlets, and the extent to which injury, more or less weighty, would be created by even small changes in this regimen.

A feature of lake regulation also difficult of appreciation, which was brought out by Mr. Gibson,** is that a system of regulation by which the discharge increases uniformly with the elevation of the lake will afford the least volume of high discharge and the greatest volume of useful flow. This is the system of regulation afforded by the natural outlets. An examination of the

* Discussion of the paper by George B. Pillsbury, M. Am. Soc. C. E., continued from April, 1930, *Proceedings*.

† Author's closure.

‡ Col., Corps of Engrs., U. S. A.; Dist. Engr., Philadelphia, Pa.

§ Received by the Secretary, March 8, 1930.

‖ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, pp. 1734 *et seq.*

¶ *Loc. cit.*, November, 1929, Papers and Discussions, p. 2424.

** *Loc. cit.*, September, 1929, Papers and Discussions, p. 1736.

results secured by Mr. Horton's suggested regulation of Lakes Michigan and Huron, in Fig. 13,* shows that while the extreme low flow is increased, the ordinary flow is decreased for a much longer part of the time. The duration curve of Mr. Horton's regulated outflow would unquestionably take the same general form as that indicated for the regulated flows of Lakes Erie and Ontario in Fig. 3.†

A third feature of regulation of the lakes, which is generally overlooked, is the limited discharge capacity of the outlets. In Mr. Horton's plan, it is proposed,‡ for example, to discharge 250 000 cu. ft. per sec. from Lake Huron when the elevation of that lake is 580.2 ft. above sea level. Assuming that Lake Erie is even 9 ft. below Lake Huron, the St. Clair River can not discharge as much as 200 000 cu. ft. per sec. when Lake Huron is at this level. The enlargement necessary to increase the discharge capacity of the St. Clair River by 25% might well cost more than the deepening of all the channels and harbors of the lakes by an amount equivalent to the maximum benefits from any suggested system of regulation. It is to be recollected that the outlets to the lakes are now, and steadily have been, discharging water just as fast as they possibly can, but that at present the lakes are nevertheless higher than some would like. The cost of enlarging the outlets—all the way down to Montreal—to a degree sufficient to afford a comfortable margin for regulation, is one of the reefs on which proposals for regulation are shipwrecked.

Mr. Horton explains§ very clearly the theoretical objections to controlling works as a remedy to the causes of impaired lake levels. It is to be observed, however, that in Fig. 10,‖ the lines, *A I* and *D E*, when considered in their entirety as extending to the virtual sills of the outlet, are not right lines but are concave downward. In any practical application of compensating works, to correct the relatively small causes that have resulted in lowering the lakes, the two curves are so nearly parallel, in the region comprised between the range of lake levels, that the distances, *K L* and *G H*, are inappreciable.

Especial attention may be invited to Mr. Ray's discussion,¶ with which the author completely concurs. It is hoped that the interesting discussions that have been presented will afford further light on this moot engineering question.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2438.

† *Loc. cit.*, September, 1929, Papers and Discussions, p. 1754.

‡ *Loc. cit.*, November, 1929, Papers and Discussions, p. 2437.

§ *Loc. cit.*, p. 2432.

‖ *Loc. cit.*, p. 2433.

¶ *Loc. cit.*, April, 1930, Papers and Discussions, p. 817.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

NIAGARA POWER

Discussion*

By Norman R. Gibson, M. Am. Soc. C. E.†

Norman R. Gibson,‡ M. Am. Soc. C. E. (by letter).§—Since the discussions were for the most part additions to, rather than criticisms of, the writer's statements, there seems little need for reply other than to thank those who have so ably contributed and to express appreciation for the information they have submitted.

Mr. Hogg's interesting charts‖ showing the magnitude and rate of daily variation in Lake Erie and Niagara River levels caused by wind, illustrate the difficulties to be solved in the problem of lake regulation. The information he has given in regard to the efficiency of Canadian plants, the growth of the Ontario power load, and the period of service and obsolescence of the plants, is an important addition to the facts concerning Niagara power.

The discussion by Colonel Keller¶ has amplified the historical account of power development at Niagara Falls and has explained clearly the provisions for the control of diversions from the river for power purposes. The writer is heartily in accord with the suggested orderly program "of combining remedial work and improvement of the Falls with the utmost increase in diversion consistent with really significant scenic values."**

The readers of this paper will be particularly grateful to Colonel Kelly for his discussion†† which submits in brief form the results of a careful study of the problem of lake regulation which was undertaken by the Joint Board of Engineers. The writer is in accord with the conclusion of this Board that it is advisable only to construct compensating works to counteract the effect of all diversions and outlet enlargements on the level of Lake Erie.

* Discussion of the paper by Norman R. Gibson, M. Am. Soc. C. E., continued from February, 1930, *Proceedings*.

† Author's closure.

‡ Vice-Pres. and Chf. Engr., Buffalo, Niagara & Eastern Power Corporation, Buffalo, N. Y.

§ Received by the Secretary, April 10, 1930.

‖ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, pp. 2413-2414.

¶ *Loc. cit.*, p. 2418.

** *Loc. cit.*, p. 2422.

†† *Loc. cit.*, p. 2424.

Mr. Johnson* has given further evidence of the economic value of water power and that these values are necessarily passed on to all consumers.

The discussions by Major Edgerton† and Mr. F. P. Williams‡ serve to indicate the viewpoint of those charged with the control of water power development under Federal and State authorities, respectively. Both point the way to early and effective co-operation in the economic development of water powers.

The writer is indebted to Mr. Stillwell§ for his further explanation of the origin of 25 cycles as the standard frequency of alternating current generated at Niagara Falls. As Mr. Stillwell, himself, played an important part in the early development of Niagara power, it is of a special benefit to have his authoritative statement on this question written into the record.

There remains, now, only the sad task of expressing sincere regret over the untimely death of Mr. Frank M. Williams. The thoughts to which he gave expression were drawn from his wide knowledge of water-power development in New York State.‖ It was the writer's good fortune to have known Mr. Williams for many years and, perhaps, it will not be considered out of place to pay tribute here to the high regard in which he was held for his accomplishments in the fields of Law and Engineering and for his charming and admirable character. The Society and the Engineering Profession have lost, in him, one of their most valued members and the writer one of his best friends.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2715.
† *Loc. cit.*, p. 2716.
‡ *Loc. cit.*, January, 1930, Papers and Discussions, p. 139.
§ *Loc. cit.*, February, 1930, Papers and Discussions, p. 355.
‖ *Loc. cit.*, November, 1929, Papers and Discussions, p. 2417.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PULVERIZING OF SEWAGE SCREENINGS AT BALTIMORE, MARYLAND

Discussion*

By C. E. Keefer, M. Am. Soc. C. E.†

C. E. Keefer,‡ M. Am. Soc. C. E. (by letter).§—The experimental work on the pulverizing of sewage screenings was concluded in October, 1926. Immediately thereafter the machine was permanently installed at the Eastern Avenue Sewage Pumping Station, Baltimore, Md., and since then has been used to grind all the screenings collected at this station. The daily quantity of material pulverized varies from 200 to 300 cu. ft.

The machine now (April, 1930) has been in successful service for approximately 3½ years. It has been necessary to install one set of hammers in the grinder since the machine was purchased; other than this, practically no repairs have been required. The grinder pulverizes practically everything that is present in sewage. However, in pulverizing rags it is necessary to introduce the material into the machine at a slow rate.

The grinder has proved its worth to the extent that plans are being prepared to install two machines for grinding screenings caught on coarse bars at the Baltimore Sewage Works.

* Discussion on the paper by C. E. Keefer, M. Am. Soc. C. E., continued from March, 1930, *Proceedings*.

† Author's closure.

‡ Prin. Asst. Engr., Bureau of Sewers, Baltimore, Md.

§ Received by the Secretary, April 7, 1930.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ESSENTIAL FACTS CONCERNING THE FAILURE OF THE ST. FRANCIS DAM

REPORT OF COMMITTEE OF BOARD OF DIRECTION

Discussion*

By Messrs. A. Floris, E. L. Grunsky, and S. B. Morris.

A. Floris,† Esq. (by letter).‡—The failure of the St. Francis Dam serves to illustrate, in a tragic way, a typical example of misinterpretation and indiscriminate use of formulas given in textbooks, without due regard to their limitations. In structural engineering such errors are quite common, but, owing to the ample safety really existing in structures the results are usually not disastrous. However, in gravity dams, this is not the case, since their actual factor of safety is probably very small.

The profile of the St. Francis Dam was composed of three distinct parts and is very similar to those proposed by Kreuter, Kresnik, and others, many years ago.§ This separation into three parts was considerably in favor among hydraulic engineers of that time. It was rather an important accomplishment when E. Link‖ showed by elementary means that gravity dams of a simple triangular shape are to be preferred. Modern practice adheres closely to the results of his investigations. The profile of the St. Francis Dam, therefore, represents an obsolete type, despite the fact that it has been used sporadically in recent years by engineers of some repute.

In the report of the Commission appointed by the Governor, it is stated,¶ among other things, that "there is nothing in the failure of the St. Francis Dam to indicate that the accepted theory of gravity dam design is in error".

* Discussion of the Report of the Committee of the Board of Direction on Essential Facts Concerning the Failure of the St. Francis Dam, continued from April, 1930, *Proceedings*.

† Civ. Engr., Los Angeles, Calif.

‡ Received by the Secretary, February 8, 1930.

§ "Ueber den Querschnitt der Staumauern," von F. Platzmann, Leipzig, 1908.

‖ "Die Bestimmung der Querschnitte von Staumauern und Wehren aus dreieckigen Grundformen," von E. Link, Berlin, 1910.

¶ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2155.

The writer believes that this statement is not only somewhat broad, but also misleading, if taken literally by engineers not thoroughly familiar with the limitations of the analysis of dams. The "accepted" theory of gravity dams is quite indefinite and, therefore, is capable of various interpretations.

Because the total pressure of water on the up-stream face of the dam is a triangle, with a base equal to the height, it is necessary, for equilibrium and economy, that the dam be also triangular in shape. The base of this triangle will depend on the specific weight of the material of the structure. In a dam of moderate height, designed in accordance with this requirement, the stresses will be maintained everywhere within permissible limits. However, this method of design does not take into consideration a possible uplift pressure. In order to avoid tension at the up-stream face of the dam, the base width must be increased, depending on the amount and distribution of this additional force. Assume, for instance, a triangular dam with a vertical up-stream slope: With the water level at the top, the base width of this dam will be a fraction, α, of the height; for example, an uplift pressure equal to the full hydrostatic head at the heel, diminishing linearly to zero at the toe, requires that $\alpha = 0.877$. For the same distribution of uplift, but only 0.75, 0.50, and 0.25, of the full hydrostatic head at the heel, $\alpha = 0.800$, 0.745, and 0.690, respectively. For a dam without uplift, $\alpha = 0.660$. The down-stream slope of a gravity dam, therefore, is a function not only of the water pressure and the weight of the structure, but of the upward action of the water as well.

The determination of the stresses in the dam due to an additional uplift pressure, linearly distributed from its maximum value at the heel, diminishing to zero at the toe, is quite simple. The total stresses produced by the combined action of water pressure, weight of dam, and uplift are equal to the stresses caused by the combined action of water pressure and weight of dam, minus the uplift pressure itself, without further computations.

The design of the St. Francis Dam did not comply with these requirements. Its steep down-stream slope reminds one of dams built in former days, when very little attention was paid to the action of uplift. The failure of the Bouzey Dam in France, on April 27, 1895, induced Maurice Levy to consider as safe only those dams in which the compressive stress at the up-stream face is not less than the full hydrostatic head.* This is equivalent to assuming a triangular uplift pressure with zero at the toe and a maximum, equal to the full hydrostatic head, at the heel. The catastrophe that followed the failure of the St. Francis Dam will, perhaps, force hydraulic engineers in this country to reconsider this problem once more with greater care than has been done in the past.

It may be true that the condition imposed by Levy is a severe one and that this extreme condition may not occur. However, gravity dams have been built recently with very steep down-stream slopes, their respective designers relying mainly upon cut-off walls, drainage wells, arch action, etc., for reserve strength. These precautionary measures against uplift, which were

* Comptes rendus de l'Académie des Sciences, Séance du 5 août 1895, and "Barrages de grande hauteur, résistant par leur propre poids." Instruction pour la préparation des projets et l'exécution des travaux, *Annales des Ponts et Chaussées*, VI, 1923, p. 30; also, reprint.

first proposed by Levy, are undoubtedly effective, but, if for unforeseen reasons, they do not function, as expected, the safety of the structure will be at stake. The presence of arch action in gravity dams is problematical and, therefore, it cannot be assumed in the design as a factor that increases safety.

No precautionary measures against uplift were provided in the St. Francis Dam. This was a grave negligence in a dam with a steep down-stream face founded on rock of a very poor quality. No grouting was done in the rock foundation, so that the water had free access below the base. This was greatly facilitated by the nature of the foundation itself.

The determination of stresses in gravity dams is based, as is well known, on the law of the trapezoid. This law presupposes that the dam is very thin as compared with its other dimensions, or, in other words, that it is a disk. This disk is formed by two adjacent plane sections perpendicular to the longitudinal axis of the dam. It is evident, therefore, that in a curved dam the thickness of this assumed disk will be variable, and will be formed by two adjacent sections passing through the center of the arch. The stresses determined by the law of the trapezoid are then quite different from those produced in a disk of equal thickness, that is, in a gravity dam straight in alignment. This difference is not on the safe side for curved gravity dams, so that the neglect to analyze the St. Francis Dam in accordance with this theory is an obvious violation of the principles of mechanics.

Several of the reports mention that the quality of the concrete in the St. Francis Dam was satisfactory. This almost unanimous agreement among the various Committees is limited, however, by some restrictions not sufficiently stressed in their respective reports. The concrete, although of "ample strength", was of medium quality and was not capable of sustaining stresses of a magnitude that was undoubtedly manifested in the St. Francis Dam, prior to its failure.

The designers did not provide contraction joints of any kind in the structure, believing apparently that such provisions were unnecessary and superfluous. It is common knowledge, confirmed by theory and experience as well as by experiment, that high tensile stresses are developed in concrete dams.* These stresses occur primarily during the cooling period, when chemical heat is generated in the cement paste. The cooling of the concrete mass produces shrinkage cracks in the structure, thus destroying the continuity of the assumed disk upon which the theoretical determination of stress is based. In such a badly cracked dam stresses are produced which, owing to their entirely different distribution, can be avoided somewhat if suitable contraction joints are introduced at proper places, so as to permit the structure to expand without causing dangerous cracks. Thus, to a certain extent, the continuity of the sections is preserved, unfavorable stresses are overcome, and a reasonable agreement between theoretical and actual stresses is reached.

As is well known, the stresses computed by the law of the trapezoid are not the greatest possible in a dam and for this reason the law is quite inadequate in considering the safety of the structure. Apparently, no attempt was

* "Temperature Variations in Concrete Dams," by A. Floris, *Hydraulic Engineer*, December, 1928, p. 736.

made in analyzing the St. Francis Dam to determine these stresses, judging from its unfavorable concave down-stream slope.

Stepping the down-stream slope of the St. Francis Dam produced a local concentration of stress in the sharp corners, which could have been avoided by omitting these steps entirely. For this reason a smooth slope is preferable.

The sliding factor in the St. Francis Dam, for elevations at which the failure began, was considerable for a foundation not properly prepared and stepped up. It was 0.6 without uplift, but 0.9 by considering uplift, two-thirds of the hydrostatic head at the heel, linearly diminishing to zero at the toe. In both cases the curvature of the dam was taken into account in computing this factor. The last value of the sliding factor is far in excess of that allowed by conservative engineers.

In conclusion, it can be stated with certainty that the failure of the St. Francis Dam was due to a combination of rather important errors in the design and construction. Its condition was aggravated materially by the bad quality of the rock, upon which it was built. Although the site has been unanimously condemned by all, as unsuitable for this type and height of dam, the writer believes that even in the case of a better rock foundation the safety would have been seriously imperiled by the faulty design of the structure itself.

E. L. Grunsky,* M. Am. Soc. C. E. (by letter).†—This report gives to the profession a digest of the conclusions of the various committees and boards of engineers and geologists which at one time or another reported on the failure. It does not, however, give the essential facts as brought out in the various reports, and these are necessary to a correct understanding of the status which must be accorded to the St. Francis Dam failure; nor does it refer to the lessons to be learned therefrom. It is not sufficient to state that the consensus of opinion is "that the dam failed because of weaknesses inherent in the foundation".‡ What the profession wants to know is in what way the foundation was inherently weak and how this weakness produced the forces which ultimately disrupted the dam. The profession is interested in the evidence that was available before the failure of the dam, and that which was disclosed following the failure; that is, the evidence which would afford to the engineer a knowledge of the actual conditions under which it was built, and why, later, it failed.

Any facts which predicated failure, or indicated unstable foundation conditions, such as cracks in the masonry and excessive or increasing foundation leakage, which would indicate movement of the structure or change in foundation conditions, should all be included as essential facts.

In many instances of engineering failure, post-mortems are valueless. The destruction is either too complete or the engineering error producing the trouble is of such character as could be called unforeseen. In the case of the St. Francis Dam, however, there was plenty of evidence on the ground (some of which has since been destroyed by the City of Los Angeles) that portions

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† Received by the Secretary, March 19, 1930.

‡ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2163.

of the dam were in movement prior to the failure, and that this movement was accelerating just before the failure occurred.

The writer made several trips to the dam. One trip was made on March 8, 1925, at which time the masonry had been constructed just slightly above the old creek level. Two trips were made after the completion of the dam when the water in the reservoir was within about 40 ft. of the top; and two inspection trips were made, following the failure, one on March 17, 1928, four days after the failure, and one in April, three weeks after the failure. The last trip was made in company with C. E. Grunsky, Past-President, Am. Soc. C. E., and Dr. Bailey Willis, Geologist, to secure data for the Grunsky report, which was made for the Santa Clara River Valley Protective Association.

The examination disclosed not only that the foundation material under the easterly end was a laminated mica schist, with a dip approximating the slope of the east abutment, but that, in addition, it was the face of an old slide which had originally moved down some hundreds of feet and finally had come to rest on the canyon floor. Its extent can be still seen on the ground, and is easily identified from the aerial photographs. This slide was in more or less stable equilibrium until the dam was built against it, and water percolating under pressure into the fissures and laminations of the more or less broken schist, lubricated the sliding planes, whereupon movement began. This movement along the cleavage planes caused a large block of the mountain side to act as a wedge against the sloping base of the east portion of the dam, thus causing uplift. Early in January, 1928, more than two months before the failure, Vernon M. Freeman, Assoc. M. Am. Soc. C. E., noticed two diagonal cracks running from the top of the dam, at angles of approximately 45°, down to the foundation at the sides. That these cracks were known and were of considerable magnitude is shown by the fact that in a large fragment of the ruins from the east end of the dam, one of these cracks could be seen, and the writer took oakum packing from the crack at the down-stream face. The breadth of this packing indicated that the crack was ½ in., or more, in width where it ran out at the abutment surface.

The diagonal cracking of the east portion of the dam is evidence that the hillside was in motion under the action of the water possibly from rains and from the reservoir, months before failure, and that this part of the dam was slowly being lifted from its foundation. That a large portion of the original slide was concerned in this movement is evidenced by long parallel cracks along the top of the remaining portion of the old slide. These cracks were at varying distances apart and occurred as far back from the edge as several hundred feet. They are not to be ascribed to under-cutting of the face, because the present face of the slide extends to the bottom of the gorge substantially on the angle of cleavage. They must have been caused by a mass movement of the old slide on smooth planes between thin layers of shattered schist.

An examination of the face of the present surface of the schist on the east hillside shows a shattered and cross-broken condition, which was the same as that of the face at the time of the construction of the dam, as shown by photographs, and as seen by the writer. The writer was told by one of the sub-foremen that during construction all dynamiting had to be stopped

because too much material would be shaken loose from the east abutment slope at each blast. The writer visited the dam at a time when concreting had been brought slightly above creek level. The east side had been stripped to cleavage planes with quite smooth surfaces, and it was apparent that concrete was to be poured against them as they stood. No cut-off trench was excavated, and no stepping into the hillside with vertical abutment faces was attempted. At the west side the next concrete pour would have reached the point where the schist and the conglomerate contacted. Men were working on the conglomerate at the time, dressing the slope for concreting, and it was noticeable that this conglomerate was readily removable with pick and shovel. On this side, also, concrete was to be poured without any trenching or stepping.

Attention is called to these conditions to show that the poor quality of the foundation was apparent even at casual inspection and that the unstable character of the mountain side at the east and the poor character of the conglomerate at the west should have caused concern at the time of construction. Had the east abutment been cut to proper shape the unsuitability of this side of the canyon for an abutment would have become apparent.

The second long diagonal crack (approximately 45°) in the west or right bank portion of the dam indicated that there had also been a lifting of that part. Since the failure, re-surveys have disclosed that swelling ground raised the long parapet-like western extension of the dam about 0.3 ft. The swelling material is the same conglomerate as that found under the west portion of the main dam.

The two diagonal cracks evidenced the fact that the dam was being subjected to uplift forces and that portions of its foundation were in motion months prior to the failure.

These diagonal cracks could not have been due to temperature or shrinkage, because then they would have been vertical; and they would have been widest at the top of the dam and smallest at the bottom. Actually, these cracks were large at the bottom and small at the top. This was shown by the oakum filling, before mentioned, which still adhered to the edges of the cracks at the time of the writer's visit to the ruins on March 17, 1928.

The writer visited the dam in 1926 after its completion, at a time when the water in the reservoir was within about 60 ft. of the spillway elevation. The most noticeable fact was that water seemed to be seeping along the foundation contacts from the creek level up to what appeared to be almost the water level. In other words, very little head was required to force water along the foundation contact surface at any point.

Furthermore, on the west side at that time the seepage was so pronounced that a trench had been dug to intercept it and deliver it into a 4 or 5-in. pipe. There was so much water flowing in the trench that the writer determined to measure it, and returned a few hours later, but found that the trench had been boarded over and that although the leakage was still visible, it could not be measured.

Another factor to be regarded as contributing to the failure of the dam is found in the sloping surfaces of the abutments. The water pressure against

the vertical face of the dam, under arch action, produced pressure against the sloping surfaces of the hillsides at both ends of the structure. The vertical component of this pressure must be assumed to have contributed, in some measure, to the destruction of the dam, particularly when the smooth-surfaced, easily slipping schist at one end and the clayey conglomerate at the other end are taken into account.

It is the writer's opinion that the forces which were acting on the dam at the time of its failure and which raised it from its base were: (*a*) The slide at the east which was wedging its way under the dam; (*b*) the swelling conglomerate at the west; (*c*) the pressure by both ends of the dam against sloping abutments, due to water pressure and the arched shape of the dam; and (*d*) hydrostatic uplift due to insufficient interception of underflow under the dam and to lack of drainage facilities both of the geologic strata under the dam and of the masonry. The writer believes that these forces finally broke the contact between the dam and its foundation, resulting in a rapid increase of the hydrostatic uplift pressure. There was thereupon a sudden tilting of the dam down stream. Such tilting would account for the apparent drop in elevation of the water surface as shown by the automatic gauge on top of the dam just prior to complete failure (attributed in most of the reports to a drop in water surface). This tilting continued with the toe as a fulcrum until a large slab about 15 ft. in thickness was spalled off from the down-stream face, and in all probability complete failure of the two ends of the dam occurred at or about the same time. The consequent relief of pressure due to the release of water from the reservoir then permitted the central portion of the dam to settle back on its foundation. That this tilting did occur is shown by the spalled off lower down-stream face of the central portion which remained upright; by the fractured condition of the down-stream or heel portion of the exposed cross-section of this central block; and by a crack extending along the base thereof at bed-rock and along the up-stream face for a considerable distance. The crack had opened up sufficiently to trap and crush a ladder with 4-in. sides when the monolith settled back into place. The movement of the dam and its complete disruption from its foundation was further shown by a check of a known point on top of this part of the dam, which showed that it had moved about 0.70 ft. to the east and south (down stream and toward the left bank).

The Commission of Engineers and Geologists appointed by the Governor of California, in its report, reference to which is made in the Committee report,* states:

"To the eastward of the standing section the water carried away a large amount of the schist not only on the side of the canyon or along the abutment, but in the bottom. Probably due to combined effect of water-soaking and undercutting, a very large and conspicuous slide has developed on the hillside on approximately the line of the eastern abutment. Material was still cascading down the face of this slide ten days after the failure, and from observation in the field it is apparent that the slide movement will continue for some time."

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2155.

The report also states (page 16):

"While as yet the manner and chronological order in which the failure of various sections of the structure occurred are not entirely certain, the present locations of the fragments from the west end indicate this as the point of initial failure which was quickly followed by progressive but rapid failure of the east end. Many of the available data indicate that the initial foundation failure occurred near or at the old fault or contact between the conglomerate and schist under the west end, and was due to the percolation of water into and through this section of the foundation, with resulting softening of the conglomerate under the dam. Either a blowout under, or a settling of the concrete at this place, or both, occurred, quickly followed by the collapse of large sections of the dam.

"It is probable that the rush of water released by failure of the west end caused a heavy scour against the easterly canyon wall at the toe of the dam. This rapidly cut away the schist including the material under the toe of the east part of the dam and caused the failure of that part of the structure. The escaping water then continued to cut away the schist from the east wall of the canyon until a maximum depth of about 30 ft. below the original foundation level was reached."

The Commission's assumption that the slide on the east side followed undercutting by water from a west side blowout shows that the significance of the cracks in the dam prior to failure, and the movement and spalling during failure, of the central still standing portion, was not understood. Practically all the reports, with the exception of the Grunsky report, overlooked these facts and assumed that a blow-out had occurred under the west part of the dam; that, in a short time, this part of the dam collapsed; and that then the water from this side swirled back and in some mysterious manner cut out the east side. The failure, however, could not have occurred in this manner. The configuration of the ground is such that had this been the case the left bank down stream from the dam would have shown the effects of the first wild dash of water under the tremendous head above the dam.

There is no evidence of this, however. Furthermore, the remainder of the dam would never have been lifted from its foundation; nor could a blowout in the east side have reversed the process and, later, cut out the west portion of the structure because, as soon as one wing or the other let go, the drop in water level would have reduced the forces on the remaining portions of the dam which would have remained solidly in place.

The writer does not believe that the central portion or the east end would have been disturbed at all had the dam failed by the under-mining of the west side. The evidence of wash in the canyon indicated that the first wild dash of water was from the east side toward the right bank, because on the right side down stream, the water dashed completely over a spur which formed the right bank of the canyon, at an elevation considerably higher than the evidence of erosion on the east side of the canyon. Furthermore, the evidence of wash in the canyon indicated that this rush of water must have occurred in such quantity and force as to tear the vegetation loose and scour the top of this spur. The only explanation that satisfies all the facts available is the lifting and tilting of the dam by the four forces, as already described.

The question as to which side failed first is of no particular consequence in itself. The preponderance of evidence pointing to a complete disintegration of the east hillside as well as the partly complete disintegration of the west side foundation can only be explained by the lifting of the dam before its collapse. Under no other hypothesis can the lifting of the entire dam off its base and its practically complete destruction under the circumstances be explained. The distribution of the fragments of the dam, also indicates an almost simultaneous breaking up of the two ends of the structure. Of the large blocks of concrete, the one carried farthest down stream is a piece from the east side; and it is possible to match each large sized west side fragment with a similar east side fragment as far down as the débris extends.

The writer has gone into considerable detail in presenting pertinent facts with comments because he feels great stress should be placed on the requirement that, in engineering structures, the engineer's judgment and interpretation of available facts and data are of as great importance as technical ability to design and skill to construct.

Engineers who have checked the design seem to agree that as a gravity structure alone on good foundation, the St. Francis Dam would have been safe; but other factors, which should have been recognized and should have received careful attention are: (1) The character of the foundation and lack of cut-off and of sufficient drainage which permitted free infiltration of water under the dam, thus causing upward pressure and softening; (2) the sliding hillside which caused additional uplift on the east side; (3) the swelling conglomerate which caused uplift on the west side; and (4) the sloping abutment surfaces which destroyed arch action, and permitted an additional uplift component on the inclined surface of the abutments as soon as movement commenced due to reservoir pressure on the face of the dam.

That neither the significance of the diagonal shear cracks which appeared months before the failure of the dam, nor the significance of the increasing leakage which at full reservoir preceded the failure, were recognized, need not be commented upon further than to call attention to the fact that, if their significance had been understood, the attendant loss of life could have been avoided.

S. B. Morris,* M. Am. Soc. C. E. (by letter).†—Shortly after the failure of the St. Francis Dam, a considerable flow of almost clear, but slightly opalescent, water was observed still seeping from the conglomerate of the westerly abutment. It occurred to the writer that it would be of value to determine the quantity of dissolved solids in such water compared to those in the Los Angeles Aqueduct water. Therefore, a sample of water leaching from the ravine immediately down stream from the westerly abutment of the dam, was taken on March 30, 1928, and another sample was taken on the same date from the Los Angeles Aqueduct. The chemical analyses which were made by Frank E. Marks, City Chemist of Pasadena, Calif., are as shown in Table 1.

* Chf. Engr., Pasadena Water Dept., Pasadena, Calif.

† Received by the Secretary, April 7, 1930.

Table 1 shows a great increase in dissolved calcium, magnesium, sulfates, chlorides, and silicates. Owing to the presence of numerous seams of gypsum in the conglomerate, it is quite natural that the greatest increase in dissolved solids should be that of the sulfate radical which was 29.4 times as great in the water leaching from the abutment as in the Aqueduct water. While, of course, it is now difficult to form any idea as to the actual quantity of dissolved solids in the larger leakage from the westerly abutment prior to failure of the St. Francis Dam, it is still interesting to note the possible effect of such leakage and the dissolved solids contained therein.

TABLE 1.—COMPARATIVE CHEMICAL ANALYSES OF WATER SAMPLES OF LOS ANGELES AQUEDUCT WATER BELOW SURGE CHAMBER OF SAN FRANCISQUITO POWER HOUSE NO. 2 AND WATER SEEPING FROM RAVINE IMMEDIATELY DOWN STREAM FROM WEST ABUTMENT OF ST. FRANCIS DAM.

Chemical content.	Sample from Los Angeles Aqueduct, in parts per million.	Seepage from St. Francis Dam Abutment, in parts per million.	Ratio.
Chloride	24.00	241.50	10.1
Evaporated solids	294.60	2 319.00	7.9
Silica	35.20	666.20	18.9
Oxides of iron and aluminum	3.00	4.00	1.3
Calcium	35.20	358.80	10.2
Magnesium	7.31	103.19	14.1
Carbonate radical	None	None	None
Bicarbonate radical	87.60	168.38	1.9
Sulfate radical	35.50	1 044.92	29.4
Alkalinity	143.60	276.00	1.9
Sodium and potassium	34.04	125.13	3.7
Hardness	117.97	1 320.08	11.2

The total evaporated solids increased from 294.60 to 2 319.00, or 2 024.4 parts per million. At this rate, a leakage of 1 cu. ft. per sec. would carry in solution 10 930 lb. per day, or approximately $2\frac{1}{2}$ cu. yd. per 24 hours, which is about 900 cu. yd. per year. By dissolving such a quantity of cementing material the strength, resistance, and imperviousness of many times this volume may be seriously affected.

These tests indicate the importance of mineral analysis of seepage water from dam foundations, particularly where such foundations are of rocks containing solvent minerals.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE BEHAVIOR OF A REINFORCED CONCRETE ARCH DURING CONSTRUCTION

Discussion*

By Messrs. Clyde T. Morris, C. A. P. Turner, and Douglas E. Parsons.

Clyde T. Morris,† M. Am. Soc. C. E. (by letter).‡—As a member of the Society's Special Committee on Concrete and Reinforced Concrete Arches, the writer has been studying the behavior of reinforced concrete arches since 1922, and has noted some of the phenomena reported by Mr. Slack.

Observations on Test Blocks.—In 1923-24, the Ohio State Highway Department constructed a multiple-arch bridge of seven spans at Conneaut, Ohio. At the request of the Special Committee on Concrete and Reinforced Concrete Arches, observations were made of the bahavior of the arches during construction.§ As a basis for comparison, a large test block of the same cross-section as the crown of the arch rib, and 6 ft. long, was cast at the same time as the crown section of the arch. This was similar to the test blocks described by Mr. Slack. This test block was exposed in all respects similar to the arch rib, and observations on 20-in. gauge lines were continued for a period of a year. Mr. Slack notes‖ a greater stress in the steel than could be accounted for by the differences in the coefficients of expansion of the two materials, and proves by cutting the steel that the stress is real. The coefficient of expansion and the modulus of elasticity of steel are definite and known. The differences must then have been in the properties of the more variable material, concrete. On the basis of the figures given by Mr. Slack, the coefficient of expansion of his concrete must have been about:

$$\frac{0.00011}{27} = 0.000\,004 \text{ per degree Fahrenheit.}$$

* Discussion of the paper by Searcy B. Slack, M. Am. Soc. C. E., continued from February, 1930, *Proceedings*.

† Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.

‡ Received by the Secretary, February 17, 1930.

§ *Proceedings*, Am. Soc. C. E., March, 1924, Society Affairs, p. 292.

‖ *Loc. cit.*, November, 1929, Papers and Discussions, p. 2290.

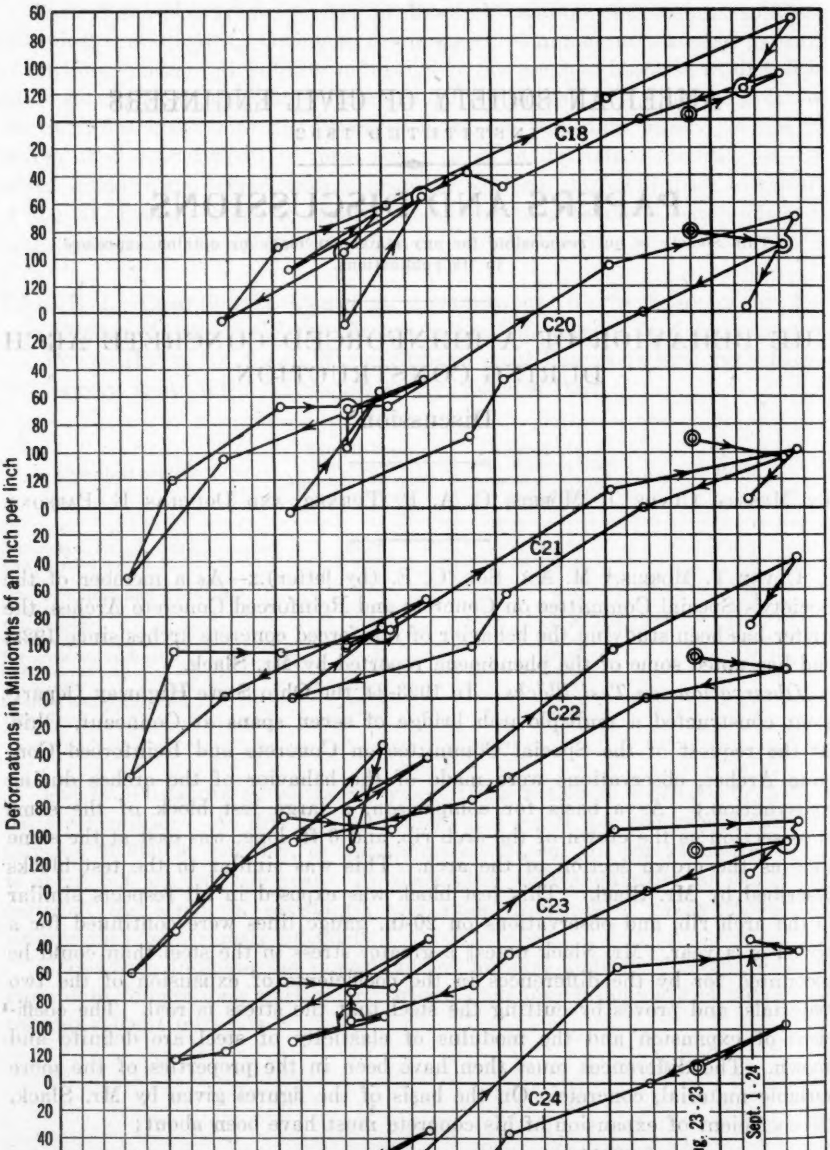

FIG. 15.—DEFORMATION READINGS ON CONCRETE TEST BLOCK AT CONNEAUT, OHIO.

Fig. 15 shows the readings taken on six different gauge lines on the test block at Conneaut. The slope of these observations indicates a coefficient of expansion of 0.000 0037 per degree Fahrenheit which agrees remarkably well with Mr. Slack's observations. While this coefficient has been attributed to temperature changes, it must be understood that it includes all climatic variations. The low temperature was usually accompanied by moist conditions and the high temperatures by dry weather. An occasional summer shower would dry off quickly and would not penetrate deeply into the concrete. The effect of rain is seen in the values of the coefficients given in Table 4.*

Pier Movements.—Observations as to rotation and settlement were made on one of the intermediate piers at Conneaut. The foundation was on a good quality of shale, and progressive settlements were observed during construction. These settlements were unequal when the pressure on the two sides of the foundation were unequal, and uniform when the pressures were uniform. The total settlement during construction was about 0.13 in. and the maximum rotation due to unsymmetrical loading was about 0.00045 radians.

These movements were small and affected the stresses in the arches only a little, but if definite movements, such as these, occur when the foundations are on rock (shale), they might well exceed safe limits when the foundations are on softer materials.

From these observations, and others made under the supervision of the Special Committee on Concrete and Reinforced Concrete Arches, it seems logical to conclude that: (1) The dead load unit pressures should be uniform over the area of the footing; and (2), careful measurements of settlement and rotation of the foundations should be made during the period of construction to guard against dangerous movements.

C. A. P. TURNER,† M. AM. SOC. C. E. (by letter).‡—This paper presents measurements of an arch rib during curing and under load subsequent to the removal of the forms. The difficulty of interpreting the measurements and drawing correct conclusions therefrom lies in the fact that during early crystallization of the cement the concrete expands. By the shrinkage grip developed between the concrete and the metal in the early stages of this expansion, the steel is stretched. If the bars are as small as ⅜ in., a tension of 7 000 to 8 000 lb. per sq. in. may readily be developed in the steel during the first two or three weeks of curing. The test blocks on which Mr. Slack made his experiments were too short to develop the amount of tension which would be developed in longer specimens.

As the concrete dies out, shrinkage takes place and the tension in the steel decreases with the shrinkage of the concrete. During the early stages of curing the partly hardened concrete has been over-strained by the tension in the steel and a state of residual strain has developed in the concrete adjacent to the reinforcing bars. Change of temperature or change of applied load will tend to dissipate or re-adjust these internal strains with some change of external form. This re-adjustment cannot be interpreted in terms

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2298.

† Cons. Engr., Minneapolis, Minn.

‡ Received by the Secretary, March 3, 1930.

of elastic stress because a portion of the change of form is unrelated to the reaction to external force.

Error in the interpretation of extensometer and telemeter readings follows the assumption that change of form of the steel invariably represents deformation attributable to elastic stress induced by applied loads. This may be far from the truth.* Reinforcing steel is occasionally kinked and afterward straightened cold before it is finally placed. It is customarily bent cold where bent bars are required. In the vicinity of sections that have been subject to such cold bending, change in form (unrelated to the applied load) may take place when the equilibrium of internal strain is disturbed by applied force.†

Although the phenomenon of residual strain has been long recognized by the physicist, it appears to be disregarded by, or unknown to, engineer investigators. The result is that accuracy and utility of a large amount of painstaking experimental work is vitiated.

The resistance of concrete is acquired in the chemical process or crystallization or hardening. Whether the chemical process of crystallization has been substantially complete or only partly so determines the deportment of the mass—whether it is elastic or semi-plastic. Plastic flow under working stresses occurs only to a negligible extent when the phenomenon of sweating in the course of curing is avoided by mixing the cement with warm water so that it does not get chilled in the early stages of hardening.

Concrete mixed with cold water at a temperature of less than 45° and subject to the frosty temperatures of early spring and fall is likely to get chilled during the early stages of hardening, so that at any time within three to six months after casting a sudden rise of temperature may cause the material to sweat, soften and deform in a plastic manner. If it has been badly chilled and if the forms have been removed only a short time, complete failure may result. Often the flow is limited to a small amount and the sweating period is likewise limited to a short duration, followed by hardening of the concrete with some permanent change of form.

Many concrete failures of years ago resulted from this cause. The safest procedure is to heat the water of the mix to a temperature of about 100 or 120° regardless of the season of the year. A cold drop of water may be rolled like a berry on a piece of cold glass. If, however, a lighted match is held under the glass and below the drop it will spread over the surface. The readiness of warm water to mix and combine with the cement as opposed to the repulsion of cold water to mixing is comparable to the experiment with the drop of water on a sheet of glass. The warm water mix produces a sticky paste not readily separated from the sand in casting and a stronger and better concrete results.

The general conclusions from many observations is that the most expert cannot determine by ordinary tests when concrete that is mixed with cold water and then chilled is substantially cured. There is no uncertainty when warm water is used in the concrete mix and reasonable care is exercised in protecting the work from cold. It does not require a freezing temperature to chill cement. Cold water and a few frosty nights are sufficient.

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1271.

† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2268.

Assuming that proper precautions have been taken to avoid chilling the cement in the early stages of hardening, how are the effects of shrinkage stresses and residual strains to be separated from elastic phenomena in experimental work? Recovery of deformations, as the test load is removed, may be interpreted as elastic stress without involving a large percentage of error. Initial readings, taken when the load is applied, may involve an error of interpretation of from 15 to 50%, dependent on the condition of the steel and the condition under which the concrete is cured.

The elastic compressibility of concrete, as Mr. Slack concludes,* readily absorbs the temperature stress that results from expansion. Experiments on viaduct sections 350 ft. long, under a drop of temperature from 40° above to 30° below zero, indicate temperature stresses less than 10% as great as those computed by the ordinary theory, and the needlessness of the multiplicity of expansion joints ordinarily used is apparent.

The measurements made by the author are of value in indicating that the change at the expansion* joints was produced largely by variation in the moisture content of the concrete. Theoretical computation of plastic flow of concrete will be found unnecessary with the proper mixture at proper temperatures. The hardening of concrete is continuously progressive when mixed with warm water and protected from being chilled. Hardening and softening may alternate in the chemical hardening of concrete which has been chilled. The softening takes place under a sudden rise in temperature and not a few failures in early concrete construction may be traced to this cause.

Because the interpretation of readings in terms of stress requires the separation of chemical changes in form, also those resulting from internal stress thrown out of equilibrium, from mechanical change caused by load, Mr. Slack's Conclusion (4)† is open to the criticism that, although observed deformations were greater than those computed from external loads, it does not follow that the elastic stress caused by the load was necessarily more than those computed.

Douglas E. Parsons,‡ Esq. (by letter).§—The many successful field tests completed since about 1910 afford conclusive evidence that under favorable conditions reliable estimates of changes in stresses in concrete structures can be obtained from deformation measurements when it is possible to alter markedly the stress conditions within a few hours or in other ways to minimize the effects of inelastic deformations. The most common type of test has been one in which the deformations produced by loadings have been measured. In making these tests it has been found advantageous to alter the loading as rapidly as circumstances would permit in order to lessen the likelihood of their being large deformations due to causes other than the loading. The load tests on the Stevenson Creek Arch Dam were made at night in order to avoid wide variations in temperature during the tests. Furthermore, a sprinkler system,

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2298.

† *Loc. cit.*, p. 2299.

‡ Chf., Masonry Constr. Section, U. S. Bureau of Standards, Washington, D. C.

§ Received by the Secretary, March 12, 1930.

supplying water to the face of the dam, was used to prevent sudden changes in the moisture content of the concrete.

In attempting to evaluate deformations in concrete over a period of approximately four months in terms of stresses, however, there are many disturbing influences. The paper gives ample evidence that the tests were carefully planned and carried out, but Mr. Slack was unable to correlate the deformations in the test blocks with those in the arch rib. In spite of the fact that he had anticipated the difficulties he would have in interpreting the measurements on the bridge, and had provided auxiliary gauge lines on members that were presumably unstressed, the data were insufficient for distinguishing between the elastic and inelastic deformations.

As indicated by Mr. Slack* the difficulty of determining stresses from measurements of deformations was due to the fact that only the total deformations occurring over a relatively long period were measured. Changes in stress, temperature, and moisture content of the concrete and plastic yield, or flow, combined their effects in producing the deformations measured. The interpretation was complicated further by changes in the properties of the concrete with age.

The need of a method for distinguishing between elastic and inelastic strains in concrete structures is apparent. Messrs. Slack and Fuller have emphasized this need in recent publications.†

A method has been suggested by W. A. Slater, M. Am. Soc. C. E., that may be useful in some cases although it has certain obvious limitations. In this method small blocks of concrete are cut out of the structure and measurments are made of the elastic recovery which takes place in these blocks as they are relieved from the forces acting when in the mass. The changes in strain accompanying the removal of the blocks may be determined by means of a hand strain gauge. The blocks may be tested after their removal to determine the relation between stresses and deformations.

A few tests have been made at the U. S. Bureau of Standards to determine the practicability of using the method in field tests of concrete structures. A concrete prism, 30 by 30 in. in cross-section and 36 in. in height, was made for the test. The prism contained one removable block near the mid-point of each of the four lateral faces. To facilitate the removal of the blocks each was cast in a trough or form. The blocks were 5 by 5 in. in cross-section and 20 in. in height. One of the forms for shaping the blocks was made of sheet metal about 0.03 in. thick; the other three were made of wall-board of three different thicknesses. All were given a thin coat of paraffin to prevent bonding with the concrete. Metal plugs were set in the concrete for use in taking strain-gauge readings in three gauge lines on each lateral face of the prism, one gauge line being in the removable block and the other two midway between the edges of the block and the prism. The principal object of the test was to determine whether the presence of the form material would appreciably

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2292.

† "Measured Strain and Temperature in a 160-Foot Concrete Arch Bridge," by Searcy B. Slack, M. Am. Soc. C. E., *Engineering News-Record*, August 29, 1929, p. 336; and letters to the Editor from Mr. Slack and A. H. Fuller, M. Am. Soc. C. E., *Engineering New-Record*, October 3, 1929.

affect the strains in the blocks and whether the blocks could be cut out without injury to the gauge lines.

The prism was placed in a testing machine and a compressive load of 600 lb. per sq. in. was applied. After taking strain readings, one of the blocks was removed by cutting the concrete along each end of the block. Although it required a cut about 5 in. long and 5 in. deep along each end of the block, these cuts were made easily with a small air-drill. The load on the prism was then released and strain-gauge readings were made. The same operation was followed in removing another block from the prism.

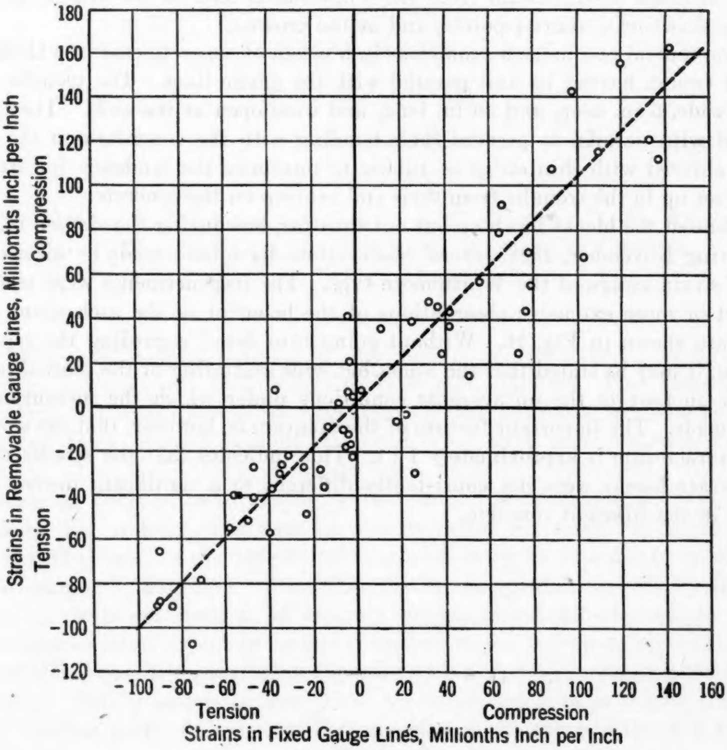

FIG. 16.

The results of the tests indicate that the blocks, when separated (except at the ends) from the prism by thin forms, were cut out without noticeable injury to the gauge lines. It was found, however, that the strain in the block cast into the sheet-steel form was only about 70% of the average strain in the adjacent gauge lines, whereas for the blocks cast into wall-board forms the strains were on the average about 30% greater than in the adjacent concrete. These results indicate that some provision should be made when using metal forms to prevent the steel from taking such an appreciable amount of stress as to affect the strain in the concrete block. They show further that wall-board,

on account of its greater thickness and lack of rigidity, is not a suitable material for such forms.

Subsequent to the laboratory tests ten removable gauge lines were established in the extrados of the arch barrel of one of the spans of the Arlington Memorial Bridge, Washington, D. C. Two fixed gauge lines were established at a distance of 10 in. on either side of and parallel with each of those to be cut out. Thus, the gauge lines were set in groups of three, all in one group being in positions expected to show equal strains. Ten groups of gauge lines were established, one group approximately 22 ft. up stream and another the same distance down stream from the longitudinal axis of the bridge, at each of the skewbacks, quarter-points, and at the crown.

The central one in each group was in a prism of concrete cast in a U-shaped metal trough having its axis parallel with the gauge lines. The troughs were 5 in. wide, 5 in. deep, and 20 in. long, and were open at the ends. They were coated with paraffin to prevent their bonding with the concrete and the ends were covered with thin strips of rubber to minimize the tendency for stresses to be set up in the troughs from their end bearing on the concrete.

None of the blocks has been cut out thus far, but during the sixteen months following November, 1928, several observations have been made by means of a hand strain gauge of the Whittemore type. The measurements were made as a part of more extensive observations of the behavior of the arch span. The data are shown in Fig. 16. Without going into detail regarding the separate causes, it may be stated that the somewhat wide scattering of the plotted points is due in part to the unfavorable conditions under which the measurements were made. The important feature of the diagram is, however, that the slope of the average line is approximately 1 : 1. This indicates that the strains in the removable blocks were not consistently different to a significant degree from those in the adjacent concrete.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

HIGH DAMS
A SYMPOSIUM
Discussion*

BY MESSRS. P. WILHELM WERNER, C. A. P. TURNER, F. W. HANNA, H. F. DUNHAM, LARS R. JORGENSEN, WILLIAM P. CREAGER, AND L. F. HARZA.

P. WILHELM WERNER,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The layout of a dam is often controlled by factors other than stability. For instance, the geological and hydro-geological formations at the dam site, and the requirements of discharging length of the crest, etc., have some bearing on this question. The desirability of obtaining a pleasing appearance§ may also influence the decision as to whether a dam should be straight or curved.

When the arching of a dam in plan is called for solely to improve the stability, it should, in the writer's opinion, preferably be as a means of obtaining an economic structure. Assuming that a gravity dam is entirely safe in itself, which it certainly is if properly designed and constructed, it seems rather unnecessary, except in special important cases, to impose upon this dam an additional condition involving expenses with a view to increasing the factor of safety. This is especially true, if the conditions are such as to give the arch action only a more or less hypothetical value, until the cantilever has been broken.

Obviously, there is a continuous transition between the pure arch and the gravity section. The question as to whether, under certain conditions, a division of the load takes place between the arch and the cantilever need not be discussed. The writer believes that this is often more or less a matter of computation, and lately it has been so treated in many instances.‖ It should be carefully noted, however, that governing factors such as shrinkage, etc., are

* Discussion of the Symposium on High Dams continued from April, 1930, *Proceedings*.

† A. B. Vattenbyggnadsbyran, Stockholm, Sweden.

‡ Received by the Secretary, January 29, 1930.

§ "The Design and Construction of Dams," by Edward Wegmann, Eighth Edition, N. Y. and Lond., 1927, p. 162.

‖ *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1191.

rather difficult to compute exactly, necessitating an ample allowance on the safe side. Any uncertainty as to the assumptions adopted for a theoretical investigation of this nature should be carefully considered in the light of actual experience on dams already constructed, etc.

The writer would not consider it safe to state that an arch of any required radius is preferable to a straight dam. It is undoubtedly true, however, that an arched dam has a larger factor of safety against ultimate failure than a straight dam of the same cross-section. The arched dam may stand by arch action, even if the cantilever has been broken, provided the abutments are capable of taking the arch thrust. In this there is, perhaps, a nucleus of a rational use of the gravity section, considering the possibility of making less provision for uplift. When doing this, however, engineers should realize clearly that they may have sacrificed some of the factor of safety against failure.

Back pressure on contraction joints or in shrinkage cracks would undoubtedly reduce the effective head on an arched dam. The question is, however, whether it is advisable, without special arrangements, to count upon it for the stability of the dam.

It might happen that full back pressure exists in one joint, but that the next joint is partly or wholly drained. This would evidently involve an unfavorable condition for the block that is subjected to the side pressure. It is questionable, therefore, whether it is consistent with a safe design to count upon the ideal conditions of full back pressure on contraction joints or in shrinkage cracks, unless special measures are taken to seal the joints and the cracks effectively at the down-stream end. It would be exceedingly interesting to know whether such measures have been applied in any case, and how they have been executed.

The writer agrees entirely with Mr. Wiley that the use of any device which offers a very short distance of travel for percolating water in the concrete, should be avoided as far as possible.* However, an effective sealing of the contraction joints at the down-stream end tends to concentrate the whole drop in pressure to the down-stream face of the dam and especially to the toe. This may cause deterioration and corrosion of the concrete in that vital part of the dam.

Furthermore, the down-stream seals in contraction joints may lead to unexpected consequences as regards the uplift. The admission of back pressure in the joints should inevitably involve a considerable increase in the allowance for uplift in and under the dam.

If the contraction joints are sealed at the up-stream face, it does not seem reasonable to assume full back pressure on the joints. Apparently, this question has a great bearing on the design. Therefore, it would be of the greatest interest and of the utmost value to have a number of completed dams visited by a committee of experts so as to determine whether or not reliance can be placed on such back pressure. In the writer's opinion it does not seem consistent with a proper design to accept an unsafe assumption on the ground that it sometimes happens to be correct.

An arched gravity dam without support from the arch and with no back pressure on the joints would have a smaller factor of safety against partial

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2324.

failure than a straight dam. The writer has also shown* (without advocating the use of the design, however) that a curved dam with a convex down-stream face under certain conditions may be more stable than a straight dam of the same cross-section. This is not a fallacy, because there is always the possibility of draining the joints, so that back pressure shall not occur. If it existed in this case, it would increase the effective head on the dam.

Uplift In and Under Dams.—In a homogeneous mass consisting, for instance, of cement or lime, there is probably no uplift, because the forces in each void (as explained by Mr. Wiley†) are opposed and cancelled by equal forces in the voids below. If this is correct, however, there must consequently be uplift at a point of perfect contact between the masonry and a rock that is free from seams, because in a tight rock there are no voids below.

As a matter of fact, there must always exist an unbalanced pressure in planes between strata of unequal porosity. Such planes are constituted by the foundation and, to a certain extent, by construction joints or other places of non-homogeneity in the dam.

Uplift conditions do not seem to be the same in concrete as, for instance, in cement or lime, Professor P. Fillunger has shown‡ theoretically that uplift exists in a mass of concrete due to the content of sand and stone. He has also derived formulas for the computation of the uplift in a dam subjected to percolating water.

C. A. P. Turner,§ M. Am. Soc. C. E. (by letter).‖—The tendency to build dams of ever-increasing height is statistically shown by Mr. Henny.¶ In general, concrete rather than masonry is best fitted for such structures because of lower cost; and reinforced concrete is preferable to plain concrete. A dam 400 or 500 ft. high presents a structural problem of the first magnitude, requiring for its most economic but safe design ingenuity of a high order, checked by the rigid application of exact elastic theory to the resistance of the materials involved.

Foundations.—For the high gravity type of dam the abnormal quantity of concrete required to follow Rankine's view of stability has been reduced by the hydraulic engineer with such increase of toe pressure that it equals or exceeds the shearing strength of softer rock and only a solid ledge of hard rock, such as granite, trap, dolomite, etc., is fit to sustain it.

The customary margin of safety of 100% or 125% in engineering structures has, in the design of high gravity dams, been reduced by the hydraulic engineer to so small an amount that dams with base widths of 66% and 85%** of the height, as compared with Rankine's 100%, are now being discussed. Why not revert to the structural engineer's idea of a liberal 100% margin of safety? Why not adopt a base of one and one-third or one and one-half times

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* *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 784.

† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2323.

‡ "Auftrieb und Unterdruck in Talsperren," by Prof. P. Fillunger, *Die Wasserwirtschaft*, No. 20-21, Vienna, 1929.

§ Cons. Engr., Minneapolis, Minn.

‖ Received by the Secretary, January 31, 1930.

¶ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2327.

** *Loc. cit.*, January, 1930, pp. 166–174.

the height so that safety against overturning would be apparent without computation or argument? Why not incline the up-stream face of the straight dam so that the resultant water pressure will push the dam down on its base, increasing its stability and shearing resistance to sliding, instead of following the unscientific practice of building a vertical up-stream face with a resultant pressure tending to overturn the dam, to raise the heel, and to crush the toe, thus illogically reducing, instead of increasing, resistance to sliding?

The carpenter millwright of years ago built timber dams applying this reasoning to his construction. The multiple-arch type of dam in which buttress walls at intervals of about 30 ft. support arched concrete plates is one resultant of this line of thought and such structures have permitted the economic development of irrigation projects that would not have been commercially feasible with the more cumbersome gravity type.

The objection urged by some against the multiple-arch type is that if a single abutment should slip or be undermined then arch after arch would fail consecutively. A further objection is urged against the thinness of the concrete plates, but this objection is based upon disintegration of porous and inferior concrete.

From the construction standpoint buttress walls in the multiple-arch dam cannot carry the great weight incident to the high dam as economically as hooped and longitudinally reinforced columns; nor can the arches be centered from abutment to abutment as inexpensively as a flat slab from column to column.

The writer's opinion as to the most economical type for the long straight high dam is a flat slab or an inclined plate design with a relatively flat up-stream slope, or batter. This slab can be supported by reinforced columns, or struts, battered to coincide with the direction of the resultant of the water pressure and vertical load. With an inclination of the plate at an angle of 40° to the horizontal the base is more than one and one-half times the height.

The quantity of concrete required for such a structure is approximately one-third that required for mass concrete of the gravity type; and when the surface rock can safely carry 20 tons per sq. ft., the quantity of concrete required would not be one-fourth as great. Offsetting the saving in part is the reinforcement and higher grade of concrete used. The margin of safety will be much more than that of the ordinary engineering structures, in order to guard against the damage and loss of life that follows failure.

Curved Dams.—This type may be divided into two classes, those in U-shaped canyons and those in V-shaped canyons. In the first class are short high dams between rock walls that are nearly vertical and in which the resistance is predominantly arch action when the height is three to four times the length of the crest. Computation on the assumption that the entire resistance is arch action will lead to a safe structure without excessive waste. On the other hand, with a V-opening and a length of crest two and one-half times the height, the resistance is predominantly a twisting resistance. The down-stream component of the normal water pressure twists the curved plate because the base cannot deflect elastically down stream so as to permit arch resistance. The cross-stream component of the normal pressure gives rise to secondary

arch action in which the cross-stream pressure is comparable to the abutment thrust of the true arch. That the resistance is predominantly a twisting resistance was demonstrated in the experiments on the Stevenson Creek Dam.* The deflection of the end quarters of the crest was up stream against pressure. No deflection occurred at the quarter-point, but the central half of the crest deflected down stream.

In masonry structures the old craftsman broke joints in his natural stone masonry, thereby attaining increased resistance approaching that of a monolithic mass. In the artificial stone of the concrete dam this practice has not been followed because of the exaggerated idea of temperature stress presented in engineering handbooks and taught in works on strength of materials.

From observation it appears that the steel stress necessary to restrain the contraction of concrete through a wide variation of temperature is less than one-tenth that computed according to handbook rules. At Moose Jaw, Sask., Canada, a flat slab structure with expansion joints between 300 and 400-ft. centers was inadvertently built with rails not cut where joints were made in the slab. These two rails (65 lb. per yd. in section) restrained the contraction of the slab of 9 000 sq. in. cross-section under a drop of 60° Fahr. After the rails were cut, the joint opened from $\frac{3}{4}$ to $\frac{7}{8}$ in. with a loud report. Ordinary computation would indicate a stress in the rails six times their yield point value and twelve times the strength of the fastenings if the contraction of the concrete were assumed to allow 2 000 000 lb. for the modulus of the concrete, which tested more than 4 000 lb. per sq. in. Considering the steel and the tension which would be required to elongate the steel $\frac{3}{4}$ in. in that length and dividing this by the area of the concrete, the tension in the concrete must have been so small that a review of the basis of ordinary computation is in order.

This method assumes without experimental verification that an elastic material under longitudinal strain expands the same amount laterally as if the longitudinal strain were lacking. Investigation of the thermo-elastic properties of matter by Kelvin indicates roughly that the shift of internal energy of vibration from kinetic to potential, or *vice versa,* represents an amount of energy twice the external work of deformation. Hence, it is logical to conclude that the temperature stress is one-half or less than one-half in the homogeneous material than that which is ordinarily computed by textbook rules. In dealing with the composite mass of concrete and metal the tension in the steel opposes the contraction of the concrete under a drop in temperature, and the steel transmits this tension to the concrete through shear, resulting in a rotation of the direction of the principal tensions 45° to the axis of the steel. It is reduced, therefore, relatively to one-half the magnitude of the restraining tension of the steel and distributed through the section in such a way that, were there diagonal reinforcements in addition to longitudinal, it would be further reduced.

As previously noted, when the two rails were sawed nearly through, the nicked section broke with a bang. The cross-sectional area thus broken, multi-

* *Proceedings,* Am. Soc. C. E., May, 1928, Pt. 3.

plied by the ultimate strength of the material when so nicked, furnishes a definite measure of the contraction stress on the 9 000 sq. in. of cross-section under a drop of temperature of approximately 100° Fahr. The total stress distributed across the cross-section would be, on this basis, less than 90 lb. per sq. in. A contraction force distributed between the longitudinal and the diagonal steel reinforcements and the concrete itself leaves a tension of less than 40 lb. per sq. in. in the concrete matrix developed by the drop in temperature of 100° Fahr., which is an amount too small either to crack or to check the concrete.

This fact accounts for the lack of difficulty which the writer has had in building flat plate dams 3 000 ft. in length without expansion joints, and bridge floors 500 to 600 ft. long without any provision whatever, under temperatures ranging from 100° above to 50° below zero. It indicates the practicability of the monolithic concrete dam when properly reinforced, so that the full resistance of a monolithic mass may be secured without serious checking or cracking of the concrete. The advantage of the well-reinforced monolith lies in its toughness, as illustrated by the example of the failure of a water tank.*

Bridge engineers a generation ago discarded the use of plain concrete because reinforced concrete presents greater toughness and dependability and because a smaller body of reinforced concrete may be used more economically. This practice of the bridge designer, which is the result of extended experience, may well be followed in the design of concrete dams for greater safety and dependability.

Unscientific computation of the maximum tension in the concrete has been presented repeatedly in Society publications, based upon the futile effort to apply the ordinary or approximate theory of flexure (suitable only for determining the maximum fiber stress in a long cantilever) to a short section of a dam having a length only a fraction greater than its thickness. Exact theory which takes into consideration shear strain is essential to practical accuracy and rational conclusions.

Resistance to external shear force is found (dependent upon the relative thickness to span length) to be several times greater per unit of cross-section with continuous and restrained beams and slabs than by the erroneous assumptions on which the complicated ordinary theory of flexure is based.

Although the correct equations for the statical equilibrium of moments was taught sixty years ago,† these principles have been so little understood or have been so disregarded by the majority of present-day engineers that computation of curved dams and flat plates supported upon columns or beams differ widely from the simple exact solutions resulting from their rigid application.

The writer's views may be summarized as follows:

1.—For the high dam reinforced concrete rather than plain concrete should be used from the standpoint of economy and safety.

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 170.

† "Natural Philosophy," by Thomson and Tait.

2.—The inclined plate has an advantage in reducing the pressure on the base to approximately one-third that of the gravity type and of reversing the distribution of maximum pressure, so that the maximum pressure occurs at the heel and the minimum pressure at the toe, thus insuring stability.

3.—Under settlement, the water pressure with the inclined-plate dam forces the column down upon its bed, and, although the slab may be cracked because of such settlement, it will still carry the load because the twisting resistance of the plate is not destroyed by checking from the top to the bottom.

4.—For the long high dam, the inclined plate is the most economical type, and its stability against overturning is apparent without computation or argument when the base is one and one-half times the height. Such a liberal base adds to the resistance against sliding, and the water pressure itself tends to increase this stability instead of reducing it as in the gravity type.

5.—For the relatively short dam on a rock base in a V-formation the curved dam may cost less than the plate structure; but if the curved dam is built the surfaces should be substantially cylindrical with flatter curvature at the ends and the sharpest curvature in the middle half. Such a dam with a length of crest two and one-half times the height should be computed for its predominant torsional resistance instead of as an arch. It should be built as a monolith and reinforced for temperature stresses.

6.—The curved dam is suitable only where the base presents a ledge of hard solid rock.

7.—The reinforced plate is suitable to dams far higher than any yet proposed. With column spacing of 30 ft. for a 500-ft. dam, the diameter of the lower columns will be approximately three-eighths to one-half their distance center to center; and the bracket heads will furnish a complete sub-arch from column to column where the pressure is greatest.

Were the dam designed with no greater margin of safety than the ordinary multiple-arch structure, the reinforcement could be treated as temperature ties for the thicker panels. Conservative practice, however, will dictate the computation of the steel on the exact theory of flexure, so that the margin of safety will be greater than that in bridge design.

In view of the enormous damage and loss of life that attends the failure of a high dam, why not make the structure so tough that it would be difficult to crack up with dynamite, instead of so fragile that a little leakage and undercutting is sufficient to cause it to crumble and fall apart, letting loose a deluge as in the case of the St. Francis failure? There is no excuse for a high dam that does not embody such toughness and safety, since the cost is much less than that of the gravity type and the work may be executed with greater facility and rapidity.

F. W. Hanna,* M. Am. Soc. C. E. (by letter).†—The Symposium on High Dams, as presented by the authors, is fundamental rather than exhaustive. It forms, therefore, basic data for elaborating more complete information from which to develop principles of dam design. In this respect it is a very useful contribution.

* Chf. Engr. and Gen. Mgr., East Bay Municipal Utility Dist., Oakland, Calif.

† Received by the Secretary, February 13, 1930.

The question of what is a high dam has not been answered in the Symposium; and, indeed, it cannot at this time be subject to answer in a single figure for any type of dam or for dams as a whole. Table 1 gives the highest dams constructed or under construction, at the present time.

TABLE 1.—Highest Dams of Various Types

Name.	Type.	Height above foundation, in feet.	Place.
Rodriquez	Ambursen	225	Mexico.
Cobble Mountain	Earth	245	Massachusetts.
Lake Pleasant	Multiple-arch	256	Arizona.
Salt Springs	Rock-fill	300	California.
Kensico	Straight gravity	307	New York.
Diablo	Arch	400	Washington.
Owyhee	Arched gravity	405	Idaho.

These data represent the present extremes of practice, and indicate to the writer somewhat of the relative future heights of the various types of dams. It now appears that the arch and the gravity types are well in the lead in this respect. The second highest gravity dam of any type in existence is the Pardee (Lancha Plana) Dam, 358 ft. from base to top, and the second highest arch dam is the Pacoima Dam, 375 ft. from base to top. The San Gabriel and Boulder Canyon Dams, both of the masonry gravity type, have been projected for still greater heights, the former for 500 ft. and the latter for 700 ft. These dams indicate the possible future trend of gravity dam heights. However, the San Gabriel Dam has been abandoned on account of a defective foundation, and the Boulder Canyon Dam has not been started and may be changed in design before it is built.

Theoretically, both earth and rock-fill dams on suitable foundations may, by proper dimensioning, be made very high; but their lack of resistance to disintegration on overtopping, their liability to destruction from leakage, and their great bulk for their height, due to increasing side slopes with increasing heights, will probably never permit them to rival in height the arch or gravity masonry dams.

The buttress types are generally built with comparatively thin sections requiring reinforcing steel for stability in design, which may limit their lives too much for high, expensive, important dams. Thin sections are also poor resisters of seepage, frost, and earthquakes. Consequently, the buttress types are not likely to be the rival high dams of the future. This, of course, does not limit their usefulness for relatively low dams.

Through the elimination of the earth, rock-fill, and buttress types for the various reasons cited, the high dams of the future are likely to be, as they are now, either the gravity or single arch type. As to which of these will be the final pinnacle of height depends on the natural formation of canyons. The gravity type will no doubt lead for wide, and the arch type for narrow, canyons. The limiting heights of these dams on hard impervious rock foundations will be determined by the allowable working stresses in good concrete;

and the dam that can come within these stress limits at least cost will ultimately be chosen. In the very high dams of these two types, stress limitations now come into play; but it is certain that stress allowances will be greatly increased with better concrete mixtures and manufacture; perhaps they may be doubled in the near future.

The arched gravity dam is superior to the straight gravity dam only in cases where the arch length is not too great, where the canyon walls will resist pressure, and where the foundation contours make it the cheaper. It is evident that no advantage results from arching the gravity type dam when the canyon side-walls are not suited to taking arch thrust. In very wide canyons no benefit is derived from arching the gravity dam because excessive deflection or buckling of the arch slice may result if the slenderness ratio is too great. When the height from foundation to crest decreases from the center toward the abutments the greater deflections of the longer cantilevers throw torsional stresses, sliding tendencies, and overturning moments into the shorter cantilevers toward the ends of the dam. The arch resistance helps the shorter cantilevers to absorb these loads. In the case of a straight gravity dam these loads can be resisted only by increased sections. That arch action actually takes place in an arched gravity dam is clear from the effects of cantilever deflection, due to water load and soaking of the up-stream face. In the computation of the stresses in the arched gravity type of dam by the usual method of taking a vertical slice of one unit width, at the up-stream face, the parallel plane theory should generally be used because such a slice between radial planes cannot slide or rotate out of place independently and this method of calculation does not give credit to the full weight of the material in the dam.*

For narrow canyons with sound rock side-walls, the arch dam is superior to the gravity dam on account of economy of materials and greater factor of safety. That the volume of the former is much less than that of the latter is evident. The factor of safety of a gravity dam against sliding may generally be only a little more than 1, and seldom more than 2, whereas that of a well-designed arch dam of the type tentatively suggested by the writer† for the Pardee Dam is 5 or 6, or more. In addition to the usual factors of safety provided for shear and compression stresses, such an arch dam has unknown additional factors of safety due to torsional resistance, to transference of stresses to the foundation by the shortest route, and to a limited degree cantilever beam action. That torsional stresses exist in arch dams has been pointed out by the writer.‡ These stresses and the corresponding resistance may be computed in each arch slice by considering the resistance to twisting of the voussoirs in it as the arch deforms under its load. The effect of arching by the shortest route to the abutments is very difficult to ascertain, but that it often exists to a considerable degree is certain. Neither the trial-load method nor the single-center cantilever method of computing the

* *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 799–808.

† *Engineering News-Record*, March 15, 1928; *Western Construction News*, May 25, 1928; June 25, 1928; and February 10, 1929.

‡ *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1246–1250.

cantilever beam resistance is very reliable because of incorrect basic assumptions. Moreover, the dry down-stream face and the wet up-stream face may produce deflections in the cantilevers that bring arch action into play and relieve the cantilevers of their loads. The fear of tension in the up-stream face of an arch dam designed as suggested is not well-founded, as cracks produced by such tension only extend into the dam a short distance and the uplift is prevented partly, if not altogether, by water soaking. This is not true of tension in the up-stream face of, and uplift in, a straight gravity dam, where increased overturning resistance is called into play. The arch dam is now, and is bound to be, a strong rival in the future for the position of maximum height in masonry dams.

Pressure grouting has been used with splendid success, both in the foundations and in contraction joints of some of the high dams now constructed, and will be used quite probably in all high masonry dams in the future. It is the writer's opinion that no high dam either of the arch or gravity type should ever be constructed without a complete grout curtain under the heel, put in under the highest practicable pressure without disturbing the rock strata of the foundation or the contact of the dam with the foundation rock.

It has also become quite general practice to construct both arch and gravity dams with vertical construction or contraction joints at intervals perpendicular to the up-stream face of the dam and sealed with metal water-stops. These joints should also be grouted under strong pressure to insure water-tightness in all high masonry dams and also arch action in the arched gravity dam and the arch dam. There is no difficulty in securing thorough distribution of the grout in the joints when they are properly designed.*

The protection against uplift in a high masonry dam by the addition of a drainage system to the grout curtain already discussed, seems to be gaining ground. The drainage system for a gravity dam should be made to consist of a cut-off trench near the up-stream toe, connected by tile pipe with a series of wells arranged to discharge into an inspection gallery of such proportions as to permit drilling. For an arch dam, drainage holes should be drilled from the down-stream face of the dam into the foundation to points near the up-stream face, thus relieving pressure.

It is also being appreciated that in high gravity dams of considerable thickness cracks are liable to occur parallel to the faces of the dam and it has been proposed to place contraction joints in such dams to control these shrinkage cracks. Whether this can be done without special treatment to care for the stresses is not yet certain because continuity of stresses from heel to toe and top to base of these dams requires continuity of resisting material. In the case of a smooth contraction joint parallel to the down-stream slope of the dam, the slab down stream from the joint may, if the joint opens, crack near its middle under its load, and fall against the adjoining concrete; or it may act as a column or strut tending to overturn the top of the dam up stream, thus producing tension on the horizontal planes.

* *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 799–809; *Engineering News-Record*, March 21, 1929.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).†—There is a measure of assurance in Mr. Wiley's discussion‡ of the effects of contraction in the curved gravity type that must be gratefully regarded by those who design high dams. The same confidence shows in the reference to uplift. If up-stream, vertical contraction cracks are not a serious weakness, and if uplift can be greatly lessened through the absence of smooth horizontal surfaces, two features in construction will seem less formidable.

While the writer's approach to the question of uplift as stated in an earlier discussion§ differed from that of Mr. Wiley, the result was not at variance with his conclusions.|| There may be noted, too, the similarity with respect to saturation of deep quarry rock and the concrete in a remaining part of the St. Francis Dam.

To avoid metal seals, the writer, in 1928, proposed the use of heating units (electrical, steam, or hot water), to be placed near each face of a dam, but not too close together. The pipes or pipe lines should be easily drained; they should be free to expand; and all should be connected to one source of heat. Used in, or prior to, cold weather the heat would be distributed and stored to offer a barrier to frost by increasing radiation. The involved principle of heat transference would be similar in some respects to the charge of a storage battery from a small unit. The slow increase in heat would be available when needed. The expense may appear favorable if set against the cost of extra masonry useful, in part, as a blanket to protect that section of a dam above the normal up-stream water elevation.

Lars R. Jorgensen,* M. Am. Soc. C. E. (by letter).†—Much discussion has taken place recently as to the exact action of a gravity dam arched in plan. Mr. Wiley favors an arched gravity dam and evidently believes that under all conditions such a dam would have the resistance of the arch in addition to its weight. His conclusion‡ that "the contraction joints of an arched gravity dam do not prevent arch action" needs considerable explanation. Most large arched gravity dams built—such as the Exchequer Dam in California—have open contraction joints in late winter. Therefore, it is physically impossible for arch action to be present in the winter when this dam supports full water pressure.

Such a dam, therefore, can not have a larger factor of safety under all conditions than a straight dam of the same section; most likely it has a smaller factor. If it had a larger factor of safety the St. Francis Dam would probably still be standing because it might not then have developed tension in the up-stream face. The Exchequer Dam has considerable tension in the up-stream face in the winter when it can receive no help from arch action. It is built on a good foundation and, therefore, apparently can take care of this tension, but it is not correct under such conditions to call it a gravity dam.

* Civ. and Hydr. Engr., New York, N. Y.

† Received by the Secretary, February 14, 1930.

‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2321.

§ *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 246.

|| *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2325.

¶ Cons. Engr., Constant Angle Arch Dam Co., San Francisco, Calif.

** Received by the Secretary, February 14, 1930.

†† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2325.

There are enough uncertainties in connection with the design assumptions and actions of gravity dams to demand a base width of not less than 75% of the height for medium high dams and considerably more for the higher dams. There is no satisfactory substitute for weight in a gravity dam.

Mr. Wiley makes the statement* that "at present the arched gravity type of masonry dam is slightly in the lead with respect to height". This is not in accordance with facts. The three highest dams, holding back the greatest depth of unsupported water load are arch dams, such as the Drac River Dam in France, 430 ft., the Pacoima Dam, 365 ft., and the Diablo Canyon Dam, 363 ft. There is no reason why this type should not or could not be built at least as high as any gravity dam on sites where it would fit, and be safer and cheaper. The safety of gravity dams, in comparison with other types, especially single arch types, has been greatly over-rated in the past. For very high arch dams it would be good engineering to use higher stresses than those now generally acceptable, say, up to a maximum compression of 1 000 lb. per sq. in., and to ensure that no concrete would go into the dam of less ultimate crushing strength than 4 500 lb. per sq. in.

This would not be at all difficult. Such a dam would have a factor of safety in excess of that of any high gravity dam that would be commercially feasible to build. Besides, in the majority of cases the arch dam would be much cheaper.

William P. Creager,† M. Am. Soc. C. E. (by letter).‡—With particular emphasis on the outlook for the future, the writer has often wondered if the theory of design, as applied to existing dams, is adequate for very high dams. The tendency is for higher dams, and it is now proposed to build a dam 700 ft. high, or nearly twice as high as the highest dam in existence. The Quebec Bridge failure taught a lesson in regard to the utilization of theories for large members or structures which have been proved adequate only for small members. Has this lesson been well learned by the designers of dams?

As a word of caution, the writer desires to call attention particularly to two features of design which, in his opinion, are in doubt for high masonry gravity dams.

The force required to move a block of stone or concrete on a like surface has been determined by tests to lie between 55 and 70% of its normal weight. Actual coefficients of friction in a large number of dams have been adopted within this range. Therefore, without consideration of the shearing resistance due to roughening of the foundations and adhesion of the concrete to the rock, a factor of safety of only 1.0 has been provided. It has been assumed, however, that this additional resistance to movement, termed "shearing resistance" in this discussion, is enough to provide sufficient margin of safety. Is this margin sufficient for very high dams? Assuming homologous triangular sections, the total shearing resistance of the foundation varies as the height of the dam, whereas the sliding force of the impounded water varies as the square of the

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2321.

† Vice-Pres. and Chf. Engr., Northern New York Utilities, Inc., and The Power Corporation of New York, Watertown, N. Y.

‡ Received by the Secretary, February 19, 1930.

height. Therefore, the shearing resistance is seven times as effective in a 100-ft. dam as it is in a 700-ft. dam. In other words, a shearing resistance of only about 30 lb. per sq. in. is necessary for a factor of safety of 2.0 in a 100-ft. dam; but, in a 700-ft. dam, 7 times 30, or 210 lb. per sq. in., is necessary. Actually, a slightly less shearing resistance than 210 lb. would be necessary owing to the greater spreading of the base of a 700-ft. dam to limit compressive stresses.

In the case of resistance to overturning, present theory requires that the resultant for reservoir full must lie within the middle third, because otherwise there will be tension in the base with the resulting tendency to open cracks and admit full head-water uplift and progressive failure by overturning or sliding. The elastic theory varies this rule somewhat, but is outside the present argument.

As many dams have been designed with the resultant close to the extremity of the middle third, it must be considered, therefore, that tension adds the necessary margin of safety. As the total resisting moment of this tension varies as the square of the height of the dam and the overturning moment as the cube of the height, the resisting moment of tension in the base is seven times as effective in a 100-ft. dam as in a 700-ft. dam.

If a dam, with the resultant at the down-stream extremity of the middle third, is assumed to have a factor of safety of 1.0, neglecting adhesion of the dam to the foundation, it requires a tensile strength of about 100 lb. per sq. in. in a 100-ft. dam and 700 lb. in a 700-ft. dam, to provide a factor of safety of 2. As in the previous case, the required tensile strength would be somewhat less than 700 lb., due to greater spreading of the base in a 700-ft. dam.

The foregoing is not intended to be an exact analysis of the subject, but is presented only to indicate that, under the generally accepted practice in the design of dams, a very large dam has a much lower factor of safety than a small one. It seems to the writer that engineers are treading on dangerous ground if they do not apply to the design of very large dams a careful consideration of those factors which, in small dams, are neglected except with the thought that they provide an additional margin of safety.

L. F. Harza,* M. Am. Soc. C. E. (by letter).†—The gravity concrete dam, whether straight or arched in plan, probably now commands the confidence of the majority of engineers to a higher degree than other types of high dams. It is open to serious doubt whether this confidence is fully justified, and whether existing rules of design can be safely extended to much greater heights without far more knowledge of behavior than is now available.

The factor of safety generally used is much less than factors used in other engineering structures, and is not sufficient to inspire confidence when one considers the many uncertainties, such as possible internal longitudinal cracking; possible tension at the heel; uncertainty as to amount of uplift along the bottom and horizontal construction joints; effect of internal pressure in the concrete; and the fact of reduction in factor of safety against sliding as the

* Cons. Hydro-Elec. and Hydr. Engr., Chicago, Ill.
† Received by the Secretary, March 10, 1930.

height of the dam increases while maintaining the same factor against overturning. The last item has been discussed elsewhere.*

The principle of keeping the line of pressure within the middle third of the base, assumes a straight line distribution of pressure from heel to toe. This is an assumption of very questionable accuracy, in a structure having the base width of a high dam, with interior temperatures differing from those at the surface, and with a somewhat flexible toe carrying the heaviest load to a foundation of perhaps not uniform resistance. This assumption ignores several uncertainties that have been pointed out by other writers. For example, it has not been possible, or at least practical, in other concrete structures to build them monolithic for a length equal to the thickness of a high concrete dam without contraction cracks, and such cracks, if approximately vertical, would reduce the effective base width of the dam. Contraction cracks might originate from shrinkage due to overheating during construction, from drying out, from seasonal temperature changes, and from stress.

In the writer's opinion, the middle-third theory is so superficial a principle that confidence in gravity dams can be based principally on reasonably satisfactory past behavior, rather than upon theory. The result is to place rules of design on inductive rather than deductive bases, and to make the design of gravity dams an experimental art, rather than a science. In fact, engineers are not justified in referring to the middle-third method as other than a "rule-of-thumb" procedure, the best support for which is not the mathematical basis, but rather the fact that few failures have occurred.

Current practice in gravity dam design is none too safe, and, in fact, a grave responsibility rests upon the engineer who goes much beyond existing precedent in the height of gravity dams without a far greater knowledge of their internal behavior, and much better control of temperatures, and uniformity of concrete, than is now possible.

However, the future is promising because it would seem that engineers are now beginning to learn something about gravity concrete dams. Intensive thought and research is now beginning to focus on this problem, so that the hope may be entertained that engineers are entering an era of research and of scientific study of dam design and the equally important factor of field control. It is probable that, as a result, the old methods will be largely discarded in favor of an analysis involving other than straight line distribution of pressure from heel to toe and other refinements.

There are possibilities of artificially controlling the placing temperatures and the curing temperatures; and several other precautions may be observed, which may assist in building dams that will perform more nearly according to the so-called theory.

In the field of power engineering there has been an ever-decreasing cost of steam energy coincident with an ever-increasing cost of hydro-electric energy; this fact is forcing upon the hydro-electric engineer the exercise of the utmost economy in the design of dams, which form such a large proportion of the cost of a hydro-electric project. Hence, the need for consideration of the merit of earth-fill dams, whether by the dry or the hydraulic method,

* See p. 1052.

combined earth and rock-fill dams, rock-fill dams, arch dams, multiple-arch dams, or other types, which may save an initial cost under certain favorable conditions.

An earth dam, well built and constructed under equally good conditions of foundation and available materials, undoubtedly compares favorably in permanency and safety with a concrete dam, when once it is successfully completed. In fact, when once completed, the earth-fill dam is subject to much less in the way of uncertainties than the writer has pointed out in connection with the gravity concrete dam; and it has given as good an account of itself in actual service.

Earth-fill or other loose-fill dams have a well recognized advantage when constructed on soft material or defective rock foundations, because they can usually be accommodated to these conditions, whereas a concrete dam would require very deep excavation, pile, or caisson supports, and often could not be built at all.

A loose-fill dam requires the protection of an ample spillway, and is, therefore, best adapted to situations where a natural ridge is available for large spillway capacity at minimum cost.

The principal uncertainty with an earth or other loose-fill dam is during the process of construction in connection with diversion of the river. Few, if any, earth dams are built, or could be built economically, if provision must be made for diversion during any stage of construction of the maximum flood which might occur on the stream in a long period of years. Construction schedules are, therefore, more important in this case than for concrete dams, and must be very carefully studied and rigidly adhered to in order to permit the work to reach a stage during a low-water season, which will insure sufficient head to divert the river through culvert or tunnel when the flood season arrives.

Earth dams may be said to be most ideally adaptable when of small volume, so that they can be built between flood seasons, thus avoiding expensive diversion work. They are well suited for small drainage areas where ample diversion provisions would not involve excessive cost. With equivalent dam site topography and foundation conditions, an earth dam would cost much more on a large stream than on a small one, because of the greater cost of diversion; or otherwise a greater hazard would be involved in its construction, if provision was not made for equally safe diversion. The same may be said of a dam of any loose material, such as a combination earth and rock-fill or a rock-fill dam.

In the case of the proposed Boulder Canyon Dam, however, the same problem of diversion is involved, because of the depth of excavation required to reach satisfactory foundation, and which excavating operation must be protected from overflow throughout one or more flood seasons. When a concrete dam is once brought up to river-bed level, however, the hazard of flood during construction is less than with dams of loose material, unless ample diversion capacity is provided for the latter.

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PAPERS AND DISCUSSIONS

CONSTRUCTION OF THE JAMES RIVER BRIDGE PROJECT

Discussion*

BY WILLIAM WILLOUGHBY, ASSOC. M. AM. SOC. C. E.

WILLIAM WILLOUGHBY,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The writer intends to confine his remarks principally to some amplification of that part of this paper pertaining to the structures over the navigable areas of the James River Project.

To meet the requirements of navigation it was necessary to provide four openings, one in each of the bridges over the Nansemond River and Chuckatuck Creek and two in the bridge across the James River. (See Fig. 1.§)

Bridges Over Nansemond River and Chuckatuck Creek.—The channels of the Nansemond River and Chuckatuck Creek were spanned with double-leaf deck-plate girder bascules of the rolling type, 110 ft., center to center of bearings. Each leaf was supported on a U-shaped pier founded on fifty-four 2-ft. square concrete piles, 80 to 85 ft. long.

The bridges are electrically operated and are equipped with solenoid service and emergency brakes. Limit switches cut out at the "open", "nearly open", "closed", and "nearly closed" positions, and an indicating light shows the interleave position of the shear locks. The bridge-operating circuits are connected in series with the warning devices and safety gates so that the bridge cannot be opened until warning is given and the gates are closed; nor can the gates again be opened until the bridge is entirely closed. The time required for operating the span is 1 min. Emergency operation by hand requires 12 min.

* Discussion on the paper by R. C. Wilson, Esq., and Herbert B. Pope, Assoc. M. Am. Soc. C. E., continued from April, 1930, *Proceedings*.

† Res. Engr., The J. E. Greiner Co., Baltimore, Md.

‡ Received by the Secretary, February 28, 1930.

§ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2358.

James River Bridge.—Nine 90-ft. through plate-girder spans across the south channel of the James River provide a horizontal clearance of 80 ft. and a vertical clearance of 25 ft. above mean high water. These spans are supported on rectangular piers on concrete piles. This opening is approximately 1 mile south of the main channel and provides access for small fishing boats to the large oyster beds on each side of the bridge.

The structure across the main channel of the James River consists of a 300-ft. vertical lift span. On each side of this span there are four 210-ft. truss spans and four 90-ft. through plate-girder spans (see Fig. 10*). The truss spans are through bridges of the Warren type. The towers of the vertical lift span are connected to the adjacent 210-ft. spans in such a way that the vertical leg of the towers rests on the channel piers and the back leg frames into the span at the second panel point. This structure provides a minimum vertical clearance of 35 ft. for a distance of 1 980 ft. The lift span over the channel provides a horizontal clearance of 250 ft., a vertical clearance of 50 ft. above mean high water when closed, and a clearance of 145 ft. when open.

The piers adjacent to the channel are rectangular in shape; the base is 28 ft. wide and 52 ft. long, supported on 190 wooden piles, each 50 ft. in length. The piers under the truss spans consist of two cylinders, connected by a web wall above Elevation +4.0. These cylinders are 10 ft. in diameter at the water-line and are flared to 23 ft. at the base. They are each supported on forty-one wooden piles varying from 40 to 50 ft. in length at all but three piers, where the piles are from 70 to 80 ft. long. The plate-girder spans rest on rectangular piers supported by twelve concrete piles 2 ft. square.

The entire site of the piers was excavated to a depth of from 6 to 10 ft. below the bed of the river, or from 30 to 40 ft. below mean high water. The piles were then driven until their tops were approximately level with the original river bed, or to refusal. They were then cut off by divers at about 8 ft. above the bottom of the excavation. In the cylinders the piles were driven in three concentric circles the radii of which were 3 ft. 3 in., 6 ft. 6 in., and 8 ft. 9 in. The center of the piles in the outside circle was thus only 2 ft. 9 in. from the location of the steel cylinder. In the channel piers the piles were spaced in nineteen rows of ten piles each and came within 1 ft. 6 in. of the line along which the sheet-piling was to be driven. It will be noted, that, due to the small amount of clearance between the piles and cylinders, or sheet-piling, it was necessary to use extreme care in locating and driving the piles so that the cylinders would go down over the pile clusters, and the sheet-piling (in the case of the channel piers) could be driven without interference.

A pile dock was built near the site of the piers from which measurements could be made for locating the piles; these docks were located by triangulation because the channel is a mile from the nearest shore. Except in the case of the three piers in which 70 to 80-ft. piles were used, the piles were driven through a steel tube 26 in. in diameter. The tube was provided with guides that fitted the leads of the pile-driver and was so constructed that it could be raised or lowered.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2371.

The pile-driver was located at the correct point, and the tube was lowered until it rested on the bottom of the excavated hole. It was long enough to reach from this point to about 5 ft. above the surface. The pile was then placed in the tube and driven by means of a follower until its top was at the desired elevation. The required length had been previously determined by driving test piles at the site of each pier. In the case of the long piles sufficient penetration was secured, before it was necessary to use the follower, to guide them down to their correct positions. The steel cylinders were then lowered over the pile clusters, guided by divers or, in the case of the channel piers, coffer-dams were constructed. In no case did the piles interfere with these operations and this good fortune was due to the care with which they were driven. A submarine seal was then placed by means of a "tremie" pipe so that the piles were embedded. In filling the cylinders, the concrete was brought to within about 15 ft. of the surface in one continuous operation. A 15-ft. seal was placed in the channel piers. The coffer-dams and cylinders were then pumped out, and the remainder of the pier was constructed in the open. The coffer-dams of the channel piers were single-wall, steel, sheet-pile dams, securely braced, and were unwatered to a depth of 30 ft. The dams were very tight and were kept dry with very little pumping.

At the site of the cylinder piers, it was necessary to construct heavy docks to which the steel cylinders were secured. This prevented the cylinders from rising as the concrete, in its fluid state, rose in the flared section.

The vertical lift span is of the direct-drive type, thus eliminating the heavy machinery required for its operation from the span. On top of each tower is a house in which is located the machinery for the operation of the span. The machinery consists of a gear train on each tower, connecting the motors directly to a ring gear on the two 11-ft. 6-in. sheaves over which the cables pass. One end of each cable is attached to the span and the other to the counter-weights. The friction between the cables and the sheaves causes the span to raise and lower when the sheaves rotate. There is no slip in the cables.

The four 40-h.p. motors (two on each tower) are synchronized in pairs. As only one pair of motors is required to operate the lift bridge, the brakes on the other pair act as emergency brakes. One pair of motors is used to operate the bridge on alternate days so as to insure the correct performance of either pair at all times.

The operation of the span is controlled from the operator's house which is on a level with the roadway through a single drum-type controller. The controller is so connected that between the position of "nearly open" and "open", "nearly closed" and "closed", which represents about 5 ft. of vertical movement, only a small amount of current can be furnished to the motors. This prevents the sudden seating of the bridge by a careless operation, which would be serious because the weight of the span and counterweights is about 2 000 000 lb. The span is also equipped with air buffers at top and bottom.

The lift span is equipped with a self-leveling device operated by motor-generator sets such that, when the bridge becomes 18 in. out of level, additional resistance is automatically thrown in series with the motor, thus slowing down the motor on the side that is leading.

In addition, in case the automatic device breaks, there is an indicating dial, 18 in. in diameter, placed at the level of a man's eyes over the control drum. This is equipped with two superimposed hands, one red, indicating the position of the far end of the span, and the other black, indicating the position of the near end. Just below the dial are two push-buttons, one painted red corresponding to the red hand, and the other black, so that the operator can throw in additional resistance in the circuit with the leading motor, thus slowing it down. It is necessary for the span to get 6 ft. out of level before the guide-shoes on it will jam. Traffic is protected by alarm bells, blinking lights, and safety gates about 200 ft. from the lift span.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

EFFECT OF TURBULENCE ON THE REGISTRATION OF CURRENT METERS

Discussion*

By Messrs. Carl Rohwer, Brehon B. Somervell, N. C. Grover, Burke L. Bigwood, G. H. Nettleton, and C. Linden.

Carl Rohwer,† Assoc. M. Am. Soc. C. E. (by letter).‡—The current meter is frequently the only available means of measuring the discharge of natural streams or artificial channels, and any information that bears on this subject is always welcomed by those who are concerned with the measurement of water. The authors have carried on extensive experiments that show the action of current meters under extreme conditions, as well as the conditions ordinarily met with in practice, and are to be commended for their work. Their summary of the comment of other investigators on the behavior of various types of current meters under different conditions, is also valuable.

The authors mention§ that the meters used in the experiments were not rated at the time of the tests because only the relative values of the observations when compared among themselves were to be considered in the experiments. In Table 2,‖ however, the velocities indicated by the meters are compared with the velocity computed from the weir discharges. Obviously, in this case, any error in the meter rating will be included in the deviation due to the obstruction in the channel. The error due to the inaccuracy in the rating of the meter is probably small in comparison with the differences observed; nevertheless, there is a definite change in the rating of meters as is shown by Table 4¶ which gives the ratings of a small Price electric meter extending over the period from 1919 to 1929. All these ratings were made under the same

* Discussion on the paper by David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., continued from February, 1930, *Proceedings*.

† Assoc. Irrig. Engr., Colorado Experiment Station, Fort Collins, Colo.

‡ Received by the Secretary, February 24, 1930.

§ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2621.

‖ *Loc. cit.*, p. 2625.

¶ Unpublished records of the Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Experiment Station.

conditions at the station of the Hydraulic Laboratory at Fort Collins, Colo. It will be noted that, in general, the changes are small from year to year, but occasionally there is a definite change. This is not due to inaccuracies inherent in rating, because the check ratings in 1921 and 1929 are almost identical.

TABLE 4.—Rating of a Small Price Electric Meter Made During the Period from 1919 to 1929, at the Hydraulic Laboratory, Fort Collins, Colo.

Date.	Equation.	Remarks.
June 7, 1919	$V = 2.201\ R + 0.025$	
November 24, 1919	$V = 2.214\ R + 0.020$	
November 26, 1920	$V = 2.199\ R + 0.052$	
July 5, 1921	$V = 2.185\ R + 0.022$	Check ratings.
July 5, 1921	$V = 2.185\ R + 0.026$	
September 27, 1921	$V = 2.176\ R + 0.027$	
March 3, 1922	$V = 2.183\ R + 0.040$	
March 31, 1923	$V = 2.196\ R + 0.035$	
June 16, 1924	$V = 2.198\ R + 0.020$	New shaft installed.
May 28, 1926	$V = 2.330\ R + 0.050$	Velocities less than 2.4 ft. per sec
May 28, 1927	$V = 2.172\ R + 0.023$	New pivot and shaft installed.
April 16, 1928	$V = 2.166\ R + 0.048$	
July 15, 1929	$V = 2.172\ R + 0.031$	Check ratings.
October 18, 1929	$V = 2.173\ R + 0.030$	

Almost without exception the variation in mean velocity caused by different obstructions in Table 2 is greater at the low velocity than at the high velocity. This seems unusual. Ordinarily, the faster the water flows, the greater is the disturbance caused by the obstruction. In the data shown in Table 2 this seems to hold true only in the case of the submerged weir.

The data plotted in Fig. 12* and Fig. 13† show that under some of the extreme conditions of the test, single-count meters give very erroneous results. This does not mean that errors of this magnitude are to be expected under conditions met with in practice. It is doubtful if the velocity would ever reverse at a point in any section that would be chosen for a current-meter measurement. It is true that the water may flow backward near the banks at certain stages at some rating stations, but as long as it remains at the same stage, it will continue to flow in the same direction. The records of all the meters show that a variation of 50% in the velocity has practically no effect on the accuracy of registration.

As shown in Table 3‡ none of the meters, except the small Ott, gives very satisfactory results when measuring the axial component of the velocity if the water approaches the meter at an angle to the horizonal plane. All the Price meters over-register, because they measure the velocity in the direction that the water is moving and not along the axis of the meter. The errors in the Price meters shown in Table 3 are the deviations due to multiplying the axial velocity by the cosines of the angle at which the water approaches the meter.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2627.

† *Loc. cit.*, p. 2629.

‡ *Loc. cit.*, p. 2631.

This is proved by Table 5 which was derived from Table 3, and shows for each angle and each Price meter the reduction of the velocity caused by multiplying by the cosine of the angle and the over-registration of the meter when compared with the axial component of the velocity. In the comparison, the means of the right and the left deviations are used.

TABLE 5.—Effect of Multiplying the Axial Velocity by the Cosines of the Angles at Which the Water Approaches the Meter.

Description.	Type of meter.	Horinzontal Angles, in Degrees.						
		10	20	25	30	35	40	45
Reduction in velocity, in percentage, due to multiplying by cosine of angle.	(Computation)	1.5	6.0	9.4	13.4	10.1	23.4	29.3
Over-registration of meter, in percentage, when compared with axial component of velocity.	Price Meter, No. 211 880	1.4	4.3	8.7	12.8	19.5	26.6	36.9
	Price Acoustic	0.7	4.7	8.0	12.4	18.2	25.2	34.8
	U. S. Geological Survey Improved Price	1.0	5.0	6.8	11.7	18.0	25.4	36.2

As shown by Table 5, the horizontal angle at which the water strikes the meter apparently has very little effect on the accuracy of registration of the velocity in the direction of flow by the Price meters until it exceeds 40 degrees.

TABLE 6.—Comparison of Current Meter Measurements by Different Meters and Methods with Weir Discharge When Current Meter Is Turned 10° to the Right both Before and After Weir Discharge Was Reduced by Multiplying by the Cosine of 10 Degrees.

Meter.	Flume width, in feet.	Depth in flume, in feet.	Mean velocity, in feet per second.	Weir discharge, in cubic feet per second.	Deviation from Weir Discharge. Method of Current Meter Measurement. Vertical Integration. With cosine correction, percentage.	Vertical Integration. Without cosine correction, percentage.	Multi-Point. With cosine correction, percentage.	Multi-Point. Without cosine correction, percentage.	Two and Eight-Tenths. With cosine correction, percentage.	Two and Eight-Tenths. Without cosine correction, percentage.	Six-Tenths. With cosine correction, percentage.	Six-Tenths. Without cosine correction, percentage.
Small Price electric,	7.980	1.742	1.076	14.96	+1.15	−0.33	......	......	......	......	......	......
with tail	7.980	1.742	1.079	15.00	......	......	+1.35	−0.13	+1.56	+0.07	+5.62	+4.07
Small Price electric,	7.982	2.118	1.796	30.36	−0.27	−1.78	......	......	......	......	......	......
without tail	7.982	2.115	1.789	30.21	......	......	0.00	−1.46	+2.79	+1.29	+4.27	+2.74
Hoff experimental, with	7.981	1.963	1.516	23.75	−3.25	−4.72	......	......	......	......	......	......
tail	7.981	1.963	1.516	23.75	......	......	−1.37	−2.86	−0.94	−2.24	−2.31	−3.79
Small Ott, with tail	7.982	2.102	1.772	29.74	+1.12	+1.65	......	......	......	......	......	......
	7.982	2.091	1.751	29.22	......	......	+1.21	−0.24	+1.60	+0.14	+5.07	+3.56

This characteristic of the Price meters has its advantages, as well as its disadvantages, because for this reason it is not necessary to hold the meter

exactly parallel to the axis of the stream in order to make it register accurately. This is not true of the propeller meters.

When the water approaches the meters at a small horizontal angle, all the meters register the axial component of the velocity with a fair degree of accuracy, as shown by Table 3. This is also shown by Table 6 which gives the results of some experiments* on cup and propeller meters using different methods of measurement made at the Bellvue Hydraulic Laboratory in 1921. In making these tests, the water was measured by the current meters in an 8-ft. rectangular channel and checked by a standard 10-ft. Francis weir constructed according to the plans of the weir used in the Lowell hydraulic experiments. Each meter was held so that its horizontal axis made an angle of 10° with the axis of the channel. The results given in Table 6 show the deviation in the measured discharge by each meter when compared with weir discharge, both with and without the cosine correction. The table shows that the results of the measurements with the small Price electric with tail and the small Ott with tail are closer without the cosine correction; the results for the Hoff meter are bettered by introducing the cosine correction, and the results for the small Price electric without tail are divided.

All the tests reported by the authors show that small changes from the normal cause relatively small errors in the registration of the meters. The large errors occur when the meters are subjected to unusual conditions. The tests indicate that, if the rating station is chosen properly, both the cup and the propeller meters will indicate the velocity with a satisfactory degree of accuracy.

Brehon B. Somervell,† M. Am. Soc. C. E. (by letter).‡—The authors have added some valuable and interesting information to the data available as to the behavior of certain types of current meters under conditions differing from the regular and normal flow of the water measured. The Board on Sand Movement and Beach Erosion appointed by the Chief of Engineers of the Army to investigate conditions at points along the Atlantic and Gulf Coasts, was faced at the beginning of its investigation with the necessity for finding a meter which would not only give reasonably satisfactory measurements with varied flow, but one which would record reversals in flow and currents of very low velocity. In the investigation of wave action on the coast it was necessary to determine certain characteristics of the waves themselves and to ascertain the combined effect of the currents caused by waves, winds, and tides.

After a canvass of the experience of the members of the Board and its Consultants, and after conferences with other Government agencies having extensive experience with current meters, the conclusions were reached that (1) there was no meter on the market that would record reversals of current every few seconds; and (2) none that would record the extremely small velocities which were expected to be found when all forces acting on the water except those of the tides were negligible. The service which the meter was required

* Unpublished records of the Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Experiment Station.

† Maj., Corps of Engrs., U. S. A.; Dist. Engr., Washington, D. C.

‡ Received by the Secretary, February 28, 1930.

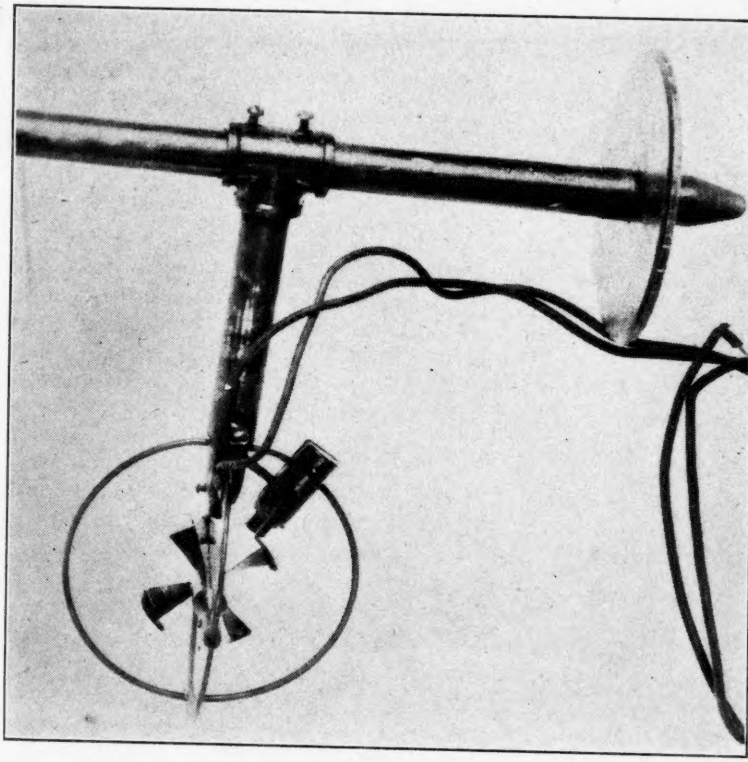

FIG. 19.—MOUNTING OF PEGRAM CURRENT METER.

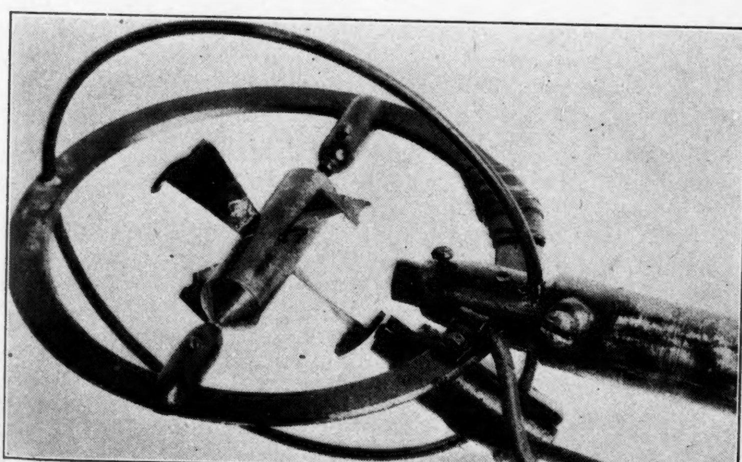

FIG. 18.—DETAILS OF PEGRAM CURRENT METER.

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to perform evidently demanded an horizontal-axis apparatus because one with a vertical axis was not adapted to measure the direction of flow of the water near the ocean beach. Various types of horizontal-axis meters manufactured in the United States as well as a French and a German meter were considered; some of them were actually put in use, to determine to what extent they might serve to measure the currents encountered. In the Ott current meter, reversals in current were shown by the alternate occurrences of long and short rings of a buzzer, denoting three and two revolutions, respectively. Inasmuch as these revolutions sometimes were very rapid and sometimes extremely slow, it was impossible to tell from the recording device whether the rings were short or long. For this reason the direction of rotation could not be established definitely, except in shallow water where the motion of the propeller could be observed. All were found to be inadequate by reason of the fact that they could not record reversals in the currents for this and other defects.

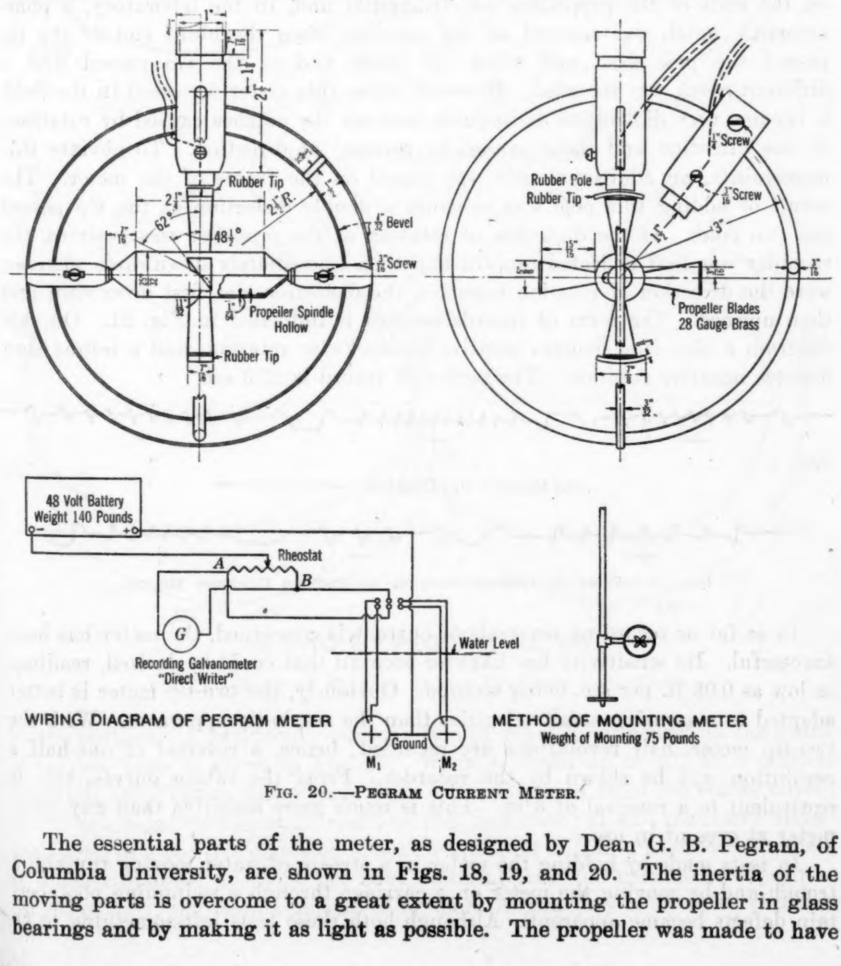


FIG. 20.—PEGRAM CURRENT METER.

The essential parts of the meter, as designed by Dean G. B. Pegram, of Columbia University, are shown in Figs. 18, 19, and 20. The inertia of the moving parts is overcome to a great extent by mounting the propeller in glass bearings and by making it as light as possible. The propeller was made to have

approximately the same specific gravity as salt water. This was done by making the hub hollow and sealing it so as to give it some buoyancy which overcame its weight and the weight of the propellers themselves. The complication of additional gears, shafts, and other devices was eliminated by recording the revolutions of the propeller electrically. As will be noted from Fig. 20, the meter is fitted with two poles insulated from the frame. The propeller has four blades. In the one-tip meter, a single blade is fitted with a rubber tip which is counterbalanced by an opposed weight on the opposite tip. In the two-tip meter, two opposed propeller blades are fitted with rubber tips. As the propeller revolves, these tips pass the poles of the meter which are connected to a recording galvanometer; the resistance of the sea water is cut down as the insulated tips pass the poles, and, with the increase in conductivity, a characteristic mark is obtained on the record of the galvanometer.

In the original meter, only one pole was used. The shape of the rubber tips on the ends of the propellers was triangular and, in the laboratory, a characteristic notch was secured on the recorder when the blunt end of the tip passed the pole first, and when the sharp end of the tip passed first a different notch was recorded. However, when this meter was used in the field, it became very difficult to distinguish between the notches caused by rotations in one direction and those caused by reversal of direction. To obviate this uncertainty, an additional pole was placed on the frame of the meter. The result of adding this pole was to cause a double deflection as the tip passed the two poles. If the direction of rotation of the propeller was positive, the recorder was first deflected upward and then immediately downward, whereas, were the direction of rotation negative, the deflection was first downward and then upward. The form of records secured is indicated in Fig. 21. On this diagram a plus sign denotes positive or clockwise rotation, and a minus sign denotes negative rotation. The period of record is 27.5 sec.

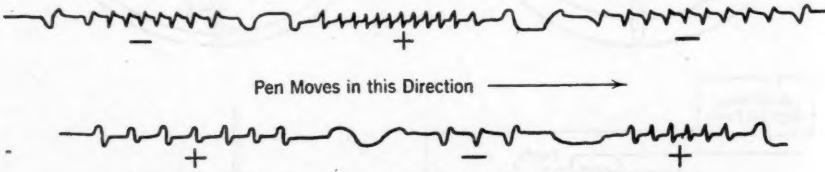
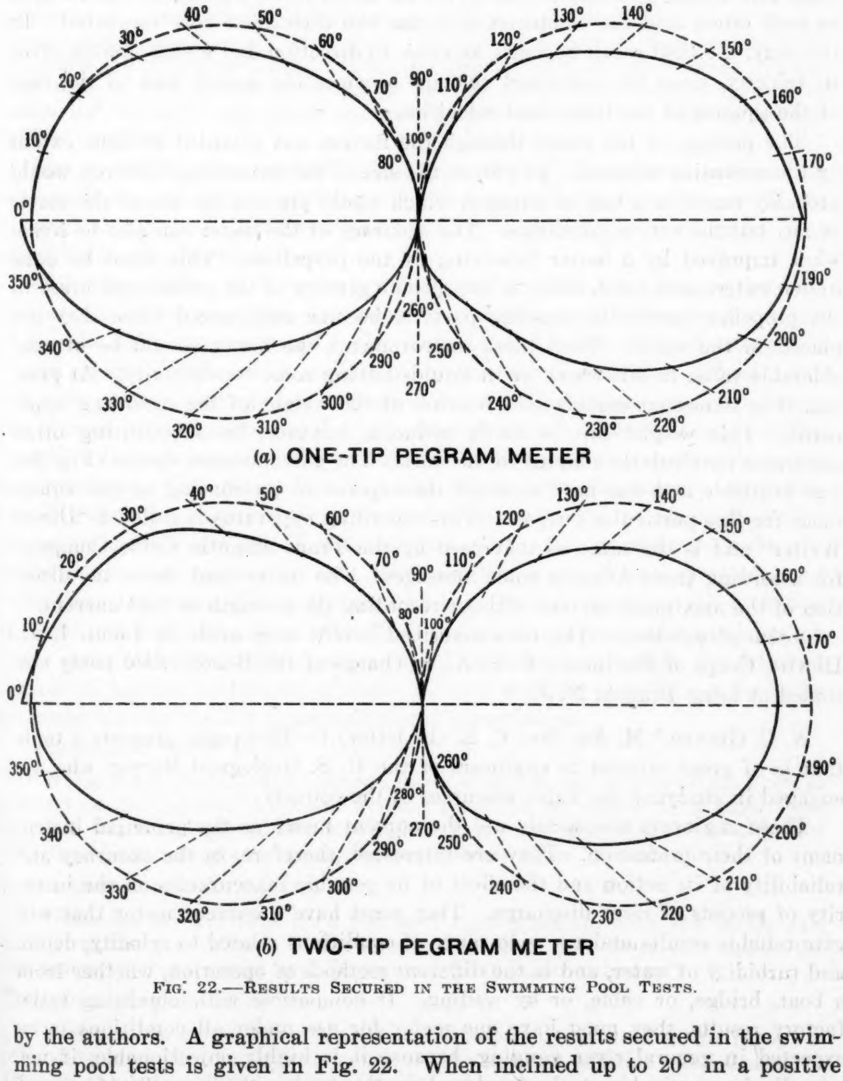


FIG. 21.—FORM OF RECORD SECURED BY PEGRAM CURRENT METER.

In so far as recording reversals of current is concerned, the meter has been successful. Its sensitivity has likewise been all that could be desired, readings as low as 0.08 ft. per sec. being secured. Obviously, the two-tip meter is better adapted to measuring small velocities than the single-tip apparatus. With the two-tip meter, half revolutions are recorded; hence, a reversal of one-half a revolution will be shown by the recorder. From the rating curves, this is equivalent to a reversal of 3 in. This is much more sensitive than any other meter at present in use.

In tests made by holding the meter in a stream of water moving through a trough and by moving the meter on a carriage through a swimming pool, certain defects became apparent. Although both these tests left something to be

desired in the way of absolute precision of measurement, they have shown that the meter is applicable to the purposes for which it was designed, and that in the measurement of oblique currents, such as might be encountered in turbulent water, its accuracy is at least as great if not greater than those described



(*a*) **ONE-TIP PEGRAM METER**

(*b*) **TWO-TIP PEGRAM METER**

FIG. 22.—RESULTS SECURED IN THE SWIMMING POOL TESTS.

by the authors. A graphical representation of the results secured in the swimming pool tests is given in Fig. 22. When inclined up to 20° in a positive direction, the Pegram meter gave little or no variation from the cosine curve; at 20° in a negative direction from 5 to 15% variation; at 40°, the meter under-registered in a positive direction about 15%, and at 40°, in a negative direction, it under-registered 20 to 30 per cent. Considering the motion in a positive direction only, the results appear to be more satisfactory than those obtained

with the meters used in the authors' experiments. These tests seem to indicate further that some change in the shape of the blades of the propeller would be desirable, in which case equally accurate readings in both positive and negative directions could be secured. In the tests made by the Board on Sand Movement and Beach Erosion, two meters were mounted on a staff, at right angles to each other, and the components in the two directions were measured. In this way, the final result becomes an error in direction and a still smaller error in velocity, since the combined velocity becomes the square root of the sum of the squares of the individual velocities.

The passage of the water through the meters was retarded to some extent by the mounting adopted. To reduce the size of the mounting, however, would probably result in a loss of strength which would prevent the use of the meter in any but the very mildest seas. The accuracy of the meter can also be somewhat improved by a better balancing of the propellers. This must be done under water, as the difference in the specific gravity of the rubber and brass in the propeller causes the rotating parts to become unbalanced when they are placed in the water. With these improvements the meter should be of considerable value in salt water, or in liquids having some conductivity. At present, it is somewhat cumbersome because of the weight of the recording apparatus. This weight can be easily reduced, however, by substituting other apparatus particularly adapted to the work. The galvanometer device (Fig. 20) was available and was used to avoid the expense of purchasing special equipment for this particular purpose. This recording apparatus is called a "Direct Writer" and is the same as that used by the Trans-Atlantic Cable Company for recording trans-Atlantic cable messages. The instrument shows the direction of the maximum current without recording the strength of that current.

Acknowledgments.—The tests described herein were made by Lieut. L. H. Hewitt, Corps of Engineers, U. S. A., in charge of the Board's field party stationed at Long Branch, N. J.

N. C. GROVER,* M. AM. SOC. C. E. (by letter).†—This paper presents a topic that is of great interest to engineers of the U. S. Geological Survey who are engaged in studying the water resources of the country.

These engineers necessarily use the current meter as the principal instrument of their profession. They are interested, therefore, in the accuracy and reliability of its action and the effect of its possible inaccuracies on the integrity of records of river discharge. They must have a current meter that will give reliable results under a wide range of conditions related to velocity, depth, and turbidity of water, and to the different methods of operation, whether from a boat, bridge, or cable, or by wading. If compatible with obtaining satisfactory results, they must have one meter for use under all conditions to be expected in general river gauging, because it is highly objectionable, if not actually impracticable, to be burdened in the field with an outfit of several current meters, each of which may be adapted only to a particular situation. The meter, also, generally should be operated by an engineer working alone,

* Chf. Hydr. Engr., U. S. Geological Survey, Washington, D. C.

† Received by the Secretary, March 6, 1930.

who is required to make all records of observations of depth and velocity and to keep all the notes related to measurements of discharge.

Early in their tests, the authors came to the conclusion, and so state,*

"* * * that a considerable degree of turbulence in the flowing water introduced large errors into the current-meter readings; but the results did not appear to be favorable for determining what modifications of type or design in the meter itself would give most promise of reducing these errors to a minimum. Furthermore, multiplying the number of such experiments did not seem to be a worthy procedure, largely because there is no existing method of measuring degree of turbulence and no standards or units in which turbulence may be expressed."

The conclusion thus reached is indicative of the difficulties experienced by field engineers as well as by laboratory investigators in connection with the measurement of turbulent water and the development of a current meter that will reliably measure such water.

Just as engineers have recognized that good leveling work can not be done in unsteady air, so they have recognized that good current-meter work can not be done in turbulent water, and as they have learned to refrain from attempting to run precise levels under conditions of unsteadiness so they should learn that they can not do first-class current-meter work in turbulent water by using any type of current meter yet devised. The greater the unsteadiness or turbulence of either air or water the greater will be the possible inaccuracies in the measurements. Unfortunately, it is impossible to define the degree of unsteadiness of either the vibrating air or the turbulent water and, as far as the writer knows, no method has been devised for expressing even approximately the different degrees of turbulence as a basis for the study of inaccuracies or of corrections for them. Thus, the authors have brought again to the attention of engineers the basic facts related to the varying effects of turbulence on meter action, but they have not attempted to define or describe the different degrees of natural turbulence of water in such manner as to form a basis for corrections to be applied to current-meter observations.

The authors state† that,

"While the data of Table 1 do not disclose the correct values of the average velocity of the current, it became evident from the angularity tests made later that the Price meter recorded too high a value and that the values obtained with the other meters are in varying degrees below the true amount."

and in their conclusions that,‡

"When meters are held on a rod in a rigid position, with the meter axis parallel to the general axis of the stream, turbulent flow invariably causes the cup meter to over-register and the screw type to under-register."

Then, they add§ the following words of caution:

"It should be remembered also that rod support was used in all these experiments, and the conclusions must not be applied without modification to cable-suspended meters. In a majority of cases, stream gauging is done with cable-suspended meters. In such suspension the tail-vanes tend to keep

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2625.

† *Loc. cit.*, p. 2623.

‡ *Loc. cit.*, p. 2634.

§ *Loc. cit.*, p. 2635.

the meter squarely facing the current regardless of any local or temporary obliquity of flow."

In these statements, the authors bring out clearly the kind of suspension used, whether by rod or cable (a condition often overlooked by laboratory investigators) and, in addition, they state what is perhaps the most important result of their tests. Although it has long been recognized that cup-type current meters over-register under conditions of considerable turbulence, it has not been so generally recognized that propeller-type meters under-register by comparable amounts. The authors have made a valuable contribution to engineering literature in making clear this important fact.

As engineers of the U. S. Geological Survey must use cable suspension in connection with a large proportion of their measurements, a comparison between the behavior of propeller-type meters and cup-type meters on cable suspension would be very desirable. However, this comparison remains now, as before, entirely unknown. It may be reasonable to suppose, as the authors have inferred, that, with all types of meter, obliquity of current may not be so important a factor with cable suspension as with rod suspension.

There is a belief apparently widely prevalent, especially among engineers of limited field experience and laboratory workers who are likely to overlook the important aspects affecting field serviceability of the meter, that the propeller (horizontal-axis) type of meter is superior in every way to the cup (vertical-axis) type of meter. Whatever the theoretical advantages of the former, however, the latter seems to have certain practical aspects of superiority that have been controlling. These are related to delicacy of action, essential accuracy, adaptability to use under a variety of conditions, sturdiness, lightness, and reliability. The authors limit their discussion to accuracy of registration. They have referred to these other aspects in a pertinent way in the final paragraph of the paper.*

The development of a current meter so that it may serve with even reasonable satisfaction under the wide variety of conditions to be found in general river measurement work is difficult. The small Price meter has perhaps been used more extensively and has received more expert study than any other current meter. As a result, in part, of this situation and, in part, of the natural advantages of the cup type, the small Price meter is now better adapted to general use than any other meter that has been developed. It is light and yet strong; sensitive and yet durable; it will measure, without great error, velocities ranging from less than ½ ft. per sec. to velocities sixty times as great. It can be used readily whether suspended by a cable or supported on a rod. It will hold its rating under rough usage; it is easily repaired, is quickly taken apart from cleaning and oiling, and is re-assembled just as quickly without change in rating. It is sensitive, although it is not equipped with ball bearings. It can be used by an engineer working without an assistant. No other current meter known to engineers of the U. S. Geological Survey satisfies these conditions.

On the other hand, this meter generally over-registers in turbulent water. Its lack of adaptability to such conditions should be recognized, and it should

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2636.

not be used in eddying or seething water or from a boat that, by a rolling motion or by rising and falling on waves, gives the meter a churning motion.

Many attempts have been made to develop a propeller type of meter for general use, and engineers look forward with hope to such a meter in the future. The writer is informally advised that engineers of the Dominion Water Power and Reclamation Service of Canada, starting with an Ott meter, are now about to produce their fourth trial meter of the propeller type to meet their needs in situations where there were difficulties with frazil ice which filled the cups of the small Price meter. They are to be congratulated in their attempts and should be encouraged to continue until they have produced a satisfactory propeller type of meter.

There are, however, many difficulties to be overcome before the equal of the small Price meter can be produced. The practically frictionless pivot bearing of the vertical-axis type of meter can not be used with the propeller type; cylindrical bearings must be substituted, if the advantages to be obtained from the overhung propeller are to be realized. Ball bearings are immediately suggested, but it is still an open question whether such bearings can be made as nearly frictionless as pivot bearings of the vertical-axis type. This will be apparent especially when it is recognized: (1) That the meters must be used in silt-laden rivers at depths greater than 50 ft.; (2) that exclusion of water from the bearings under these conditions is probably impossible; (3) that silt, often gritty, will be forced into them to interfere with the action of, and to increase the friction in, the bearings; and (4) that such bearings are not easily cleaned. In addition, the propeller must be overhung if there is to be no obstruction in front of it. This, of course, entails the use of a shaft of considerable strength and size, thereby increasing the friction and decreasing the sensitiveness of operation of the meter. The wheel must also be perfectly balanced and must remain so under ordinary usage, or it will not operate evenly in water that moves at small velocity, but will tend to stop when the heavy part of the propeller is below the axis. By contrast, a meter of the cup type, equally eccentric on its axis, would continue to revolve in water moving at the same velocity. Even a minor injury to the vanes of a propeller type of meter will change the rating, whereas experience has shown that injured cups of the small Price meter can be pressed into approximate shape in the field and the use of the meter continued without change of rating.

Because of inaccuracies in the operation of current meters in turbulent water, engineers have for many years specified that sites for river-gauging stations must be so selected that current-meter measurements will be made in water that is free from cross-currents, boils, and eddies; and, ordinarily, river measurement work is performed under selected conditions that are known to give reliable results. This situation is recognized by the authors in their reference* to "the usually good velocity conditions found at most gauging stations." That the results obtained by engineers of the U. S. Geological Survey are generally reliable has been shown many times by comparisons with measurements of discharge made by other current meters and in other ways.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2612.

The latest comparisons of this kind that have come to the attention of the writer are the following:

A comparison between measurements by means of the salt-velocity method and the small Price current meter was made in a canal at Bellows Falls, Vt., on August 16 and 17, 1929. The salt-velocity determinations were made by C. M. Allen, M. Am. Soc. C. E., and the current-meter measurements, by H. B. Kinnison, Assoc. M. Am. Soc. C. E. This statement relative to the comparison has been abstracted from an unpublished report by Mr. Kinnison. The width of the canal at the surface is 100 ft., and at the bottom 35 ft.; the side slopes are 1 : 1¼; and the depth of water was about 28 ft. The maximum velocity of water observed during the tests was 6.56 ft. per sec. The current-meter measurements were made within the stretch in which the salt-velocity determination was made and about 35 ft. down stream from the pipes used to inject the salt water. There was, therefore, some turbulence in the section where the current-meter measurements were made, due to the pipes. Two salt-velocity determinations were made by Professor Allen in connection with each current-meter measurement of discharge, one at the beginning and one at the end of each measurement. The discharge by the salt-velocity determination is the average of the two determinations thus made. The first comparison was made on August 16 when 1 hour and 41 min. were required for the current-meter measurement. The average of the two salt-velocity determinations was 9 780 sec-ft.; the discharge as determined by current meter was 9 896 sec-ft.; the excess shown by the current meter over the salt-velocity method was 1.19 per cent.

The second comparison was made on August 17 when 1 hour and 18 min. was used in making the current-meter measurement. The mean of the two determinations by the salt-velocity method was 8 587 sec-ft., and the discharge as determined by the current meter was 8 612 sec-ft. The excess shown by the current meter over the salt velocity method was 0.29 per cent.

A comparison of measurements by the salt-velocity method and by the small Price current meter was made in 1924 by S. B. Soule, District Engineer, U. S. Geological Survey, Madison, Wis. This comparison was made in the Mississippi River at St. Paul, Minn., between gate-openings of the Ford Power Plant. The gate-openings were rated by Professor Allen by the salt-velocity method. Current-meter measurements were made by Mr. Soule in the river about 1 200 ft. down stream from the power plant. The mean depth of water in the measuring section was about 6.5 ft. Two comparisons were made on December 19, and two on December 20, 1924. In the first comparison the discharge obtained from the gate-openings was 1 927 cu. ft. per sec., and by the current meter, 1 975 cu. ft. per sec.; the excess by current meter was 2.5 per cent. In the second comparison, the discharge from gate-openings was 2 290 cu. ft. per sec., and by current meter, 2 342 cu. ft. per sec., an excess by current meter of 2.3 per cent. In the third comparison, the discharge by gate-openings was 1 990 cu. ft. per sec., and by current meter, 2 033 cu. ft. per sec., an excess by current meter of 2.2 per cent. In the fourth comparison, the discharge by gate-openings was 1 668 cu. ft. per sec., and by current meter, 1 666 cu. ft.

per sec., an excess by current meter of 0.1 per cent. The report of these comparisons has been abstracted from an unpublished report by Mr. Soule.

Comparisons have been made by Mr. Kinnison between calibrated weir measurements at the Alden Laboratory, Worcester, Mass., and the small Price current meter as follows:

(1) Discharge by weir, 1.74 cu. ft. per sec.; by current meter, 1.75 cu. ft. per sec.; an excess for the current meter of 0.57 per cent.

(2) Discharge by weir, 25.05 cu. ft. per sec.; by current meter, 24.83 cu. ft. per sec.; a deficiency for the current meter of 0.88 per cent.

(3) Discharge by weir, 31.469 cu. ft. per sec.; and by current meter, 30.83 cu. ft. per sec.; a deficiency for the current meter of 2.0 per cent.

It would be unfortunate if engineers were led by this paper to lose confidence in the reliability of the current meter because, in the writer's opinion, there is no occasion for loss of confidence. The authors have created artificial turbulence in order to show clearly the dangers resulting from the use of current meters in turbulent water and to compare the behavior of meters of different types. They should perhaps place greater emphasis than they have on the fact that their observations of meter operation in water that is not turbulent confirm the essential accuracy of the action of current meters of both standard types when operated under proper conditions.

Moreover, there is no other method for obtaining reliable records of discharge as expeditiously or as cheaply as by use of the current meter. Other methods, which may be used in tests of water-wheels, or otherwise, where accuracy is essential and where a stretch of smooth-flowing water is not available for current-meter work, are not adapted to general river measurement because they are often more difficult to apply and always much more costly in their application.

Burke L. Bigwood,* Esq. (by letter).†—The experimental data presented in this paper apparently prove conclusively that, under certain conditions of flow, results of stream measurements that are obtained by use of the current meter may be grossly in error. Even without such experimental proof, trained users of the current meter would be cognizant of the existence of such inaccuracies in its operation. The concrete evidence submitted by the authors confirms beliefs heretofore based on general observation and reasoning only.

The current meter is in widespread use at the present time, as it has been for many years. In the United States and in Canada the cup-type meter is generally preferred. The U. S. Geological Survey uses the small Price meter almost exclusively in its vastly extensive activities in the field of general stream-gauging work.

It is the general opinion that at a first-class river-gauging station this type of meter produces an accurate record of the flow of the stream. This is assuredly the opinion of the personnel of the U. S. Geological Survey. Practical tests have been made which bear out this belief. This meter has been tested against accurate weirs of all kinds, against volumetric methods of meas-

* Dist. Engr., U. S. Geological Survey, Hartford, Conn.

† Received by the Secretary, March 13, 1930.

urement, against Venturi meters, and against salt-velocity and other so-called chemical methods of measurement. An imposing array of evidence could undoubtedly be assembled to prove that this meter, properly operated, gives results well within required limits of accuracy. This precision in results has been brought about and maintained by the practical elimination of those conditions of flow which produce the errors in operation of the meter.

First-class river-gauging stations contain as a most essential element a "smooth" measuring section. By this is meant a section of the stream channel through which the water flows with a minimum of turbulence and horizontal angularity. It would be expected that any type of meter would produce good results at such a station, but it is the writer's opinion that the Price meter has one great advantage, indicated by the results of the authors' experiments.

Through the selection of "smooth" measuring sections the adverse effect of turbulence and of lines of flow approaching the meter at angles from the right or left has been practically eliminated; but owing to the character of river beds in general, it is impossible to eliminate the effect of roughness of the bed of the stream on an observation to determine the velocity near the bottom. It is the writer's opinion that the usual rocky or boulder-strewn bed creates waves in the lines of flow near the bottom of the stream. This does not amount to turbulence since lines of flow above, toward the surface, are not disturbed by this action. A meter suspended in water flowing in this manner is acted upon by lines of flow approaching at angles from above or below. Under this condition, the experiments outlined in the paper* prove that the Price meter gives the best results.

All meters give least reliable results under conditions of turbulence and horizontal angularity. Having eliminated these conditions by the proper selection of gauging sections, the remaining conditions are such that, in the writer's opinion, the Price meter gives the most reliable results, mainly owing to its ability to measure accurately the wave flow near the bottom of the section.

G. H. NETTLETON,† ESQ. (by letter).‡—This paper covers important ground from the meter user's point of view. The experimenters are to be complimented on the thorough manner in which the tests were carried out but the writer cannot agree entirely with their findings for reasons here given.

The fact that the bucket type of current-meters has a tendency to over-register and those of the screw pattern to under-register when they are acted upon by very turbulent streams of water, has been long recognized. In this paper much interesting work has been described that bears out this contention; but it appears that, during the actual tests, meters were subjected only to turbulence of a most violent nature. Such conditions may be exaggerated with the definite purpose in view of ascertaining some weakness in the behavior of an instrument, but the fact must be borne in mind that conditions may be exaggerated to such an extent that they bear little or no relation to actual stream

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2632, and Fig. 16, p. 2634.

† In Chg., Dominion Govt. Current Meter Rating Station, Dominion Water Power and Reclamation Service, Calgary, Alberta, Canada.

‡ Received by the Secretary, March 17, 1930.

conditions at average measuring sections. This latter contention would appear to be the case in the experiments described by the authors. The turbulence was of such a degree that meters were forced to operate under conditions for which they were not designed and which would not be encountered at what may be described as even the more poorly selected measuring sections. Further, the artificially produced turbulence was of such a high magnitude that it was out of proportion to the mean velocity of the stream, namely, from 1 ft. to 5 ft. per sec.

It is difficult to conceive any conditions which might exist in the open channel of a river having much greater velocities than those used in the tests, which would produce such abnormal turbulence, and it would be even less true of a selected measuring section. Rapids or water-falls might be responsible for such violent agitation and they would cause vortices, etc., but no engineer would ever consider attempting a measurement in a stream at such locations. In support of the writer's contention that the artificially produced turbulence and the conditions under which the meters were used in the tests were not applicable to actual field measurements, he would draw attention to the following points.

The fixed paddle-wheels (Fig. 11*) were apparently only 6 ft. up stream from the meter and were of such a size that they actually masked it to a considerable extent, and must have almost, if not entirely, prevented the natural tendency of an unobstructed flow to smooth out deviating threads of current. In this case the angle at which the water was caused to impinge on the meters was very much in excess of that encountered in any river, and unobstructed flow could hardly have existed anywhere in the cross-section of the experimental flume at the meter section. Further, in the test with the vertical paddles (Table 1†), there was a gap of 3 in. between their bottom edges and the bed of the flume. In the case of one paddle this would have the effect of creating a high velocity undershot current which would turn sharply upward from the flume bed owing to the low pressure area at the back of that paddle. From the viewpoint of producing violent turbulence, the arrangement was excellent, but it would appear impossible to believe that such conditions existed at measuring sections.

Again, rolling vortices appear to be treated as though they were moving along and resting on the bed of a river in the same manner in which a wheel rolls over a road. In fact, in some tests, what may be described as the negative or low velocity side of such a rolling body of water was apparently given a value approximately equal to the mean velocity with which the water was flowing down the flume. This is anything but true of river conditions because such a body of water as a whole is moving down stream and the negative or low velocity side has a forward and not an up-stream velocity in an open channel. These changes in velocity affect the meter in a river as current pulsations and, as is conceded by the authors, pulsations in the flow of a stream do not cause a meter to register incorrectly. It is true that in the experiments pulsating currents did cause some difference in the recording of instruments

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2622.

† *Loc. cit.*, p. 2624.

of the cup and screw type, but the difference was accounted for by the artificially produced effect of the current changing from a positive to a negative value, or from flowing down stream to up stream.

If a meter was freely suspended by cable in a river-measuring section where such conditions existed, the instrument would naturally face the direction of flow and, consequently, at times, would be pointing up stream and at others down stream. Such conditions have never been encountered by the writer even in turbulent mountain rivers, but the flowing water always had a down-stream or positive value, even though the velocity might vary from time to time. Experiments carried out by the writer showed that what may be described as pulsating currents, that is, currents the velocity of which rose and fell above and below a mean velocity, did not cause either a meter of the screw or cup type to register incorrectly. The value of the pulsations was exaggerated purposely so that the difference in rise and fall of the velocity was considerably in excess of anything of the nature that had been experienced in field work, but at no time was the velocity given a negative or up-stream value. Furthermore, numerous measurements of different rivers have been made, using a cup and screw type of meter at the same time. These rivers were representative of those found in Alberta, Canada, in the foot-hills of the Rocky Mountains. In all cases the difference between the mean velocity recorded by each type of instrument was of such small value that it was negligible, and, further, the cup meter did not always show the greater velocity; frequently the screw meter gave the higher figure.

Although it is agreed that violent disturbance of a stream will usually cause a cup type of meter to over-register and a screw pattern to underestimate true velocity values, yet it is considered that at the average selected measuring sections, instruments of either pattern will give reliable data to within the accepted degree of accuracy for stream measurement work. Furthermore, the experiments, as described in the paper, were conducted under abnormal conditions of flow and turbulence which made it impossible for the current meters to function either as they would in the field, or under the conditions for which they were designed.

C. LINDEN,* ESQ. (by letter).†—The investigations described by the authors have added valuable data to the information pertaining to current meters and current meter action; but like all previous tests these have failed to bring definite results or to end the controversies which have prompted the studies.

Investigations are doomed to failure when it is assumed that the forces which actuate the meter represent only the true velocity. Under ideal conditions this may be very nearly true, but in turbulent waters the flow represents a combination of forces of which the velocity may be only a small part. Judging from the design of some well-known meters it must have been assumed that anything which may be rotated by the forces of flowing water may be rated and thus may become a current meter. If streams proceeded as parallel fila-

* Secy., Scientific Instrument Co., Berkeley, Calif.

† Received by the Secretary, March 26, 1930.

ments, this assumption would be true, but such is not the case, and meter rotation is not a definite product to which correcting factors may be successfully applied. Meter rotation is the result of forces of irregular origin.

Other erroneous assumptions which have been made are that meter work can be properly done in both clear water and in water filled with floating débris, and that meters should be rated under field conditions similar to those in which they will be required to operate. This would be highly desirable, but it is impractical because of the ever changing condition of both direction and velocity. No mechanical meter can produce accurate results unless the flow and operating conditions are identical with those under which it was rated. Under all other conditions the results are only approximate. The object of meter studies is, however, to produce an instrument that will approach accuracy as nearly as possible when operated under field conditions. This has proved to be a much more difficult task than was at first anticipated.

It is not possible to establish comparable ratings from meters of various types when simply held in streams of varying turbulence. The importance of maintaining a constant and definite relation between the position of the meter axis and the direction of flow does not seem to have been fully recognized in past investigations.

Theoretically, stream velocities can be measured as a function of the velocity force, but it is too much to assume that true stream velocities can be secured by mechanical means. In order to do so, the mechanical device must be capable of differentiating between true velocity forces and other influences. It is not likely that irregular stream conditions, in which the re-acting forces are caused by the influence of the meter itself, will result in consistent meter action. In turbulent or extremely irregular water, such as is found in the tail-races of power plants, it is too much to expect a meter to distinguish between true velocity forces and the many other disturbing and re-acting forces. Mechanical devices for stream-flow measurements have no place under these conditions, although many investigators apparently depend on records obtained under such conditions for stream flow data.

Any mechanical device or meter will act in accordance with the forces applied, and identical forces will give identical readings. It has been suggested that meters should be calibrated or rated under field conditions in order to have stream conditions the same as those in which the meter is to be used; but since the calibration of a meter involves the establishment of a true ratio between flow velocity and meter rotation, it presupposes definitely known velocities and a constant flow direction. In the field neither of these essentials is present and, therefore, meter ratings under these conditions are impossible. Current-meter rating consists of more than exposing a meter to stream flow or causing the meter to move through still water. The meter must be held in a stable, least flow-obstructing position during the rating process.

Some stations do not give identical ratings even for the same meter on repeated readings. This is due to limited cross-sections of the rating channel, instability of the meter support, and inaccurate measurement of both time and distance of travel.

The rating flume used by the authors was evidently not of standard dimensions. In order to effect an ideal flow the channel should be not less than 60 ft. long, with a boarded section, the full width of the canal. When the flume is less than the full width of the canal, it should exceed 60 ft. in length, in order to overcome, completely, the entrance eddies. A cross-section of 3 by 4 ft. is insufficient even for meters of the small Price type where, under most favorable conditions, the flow-obstructing area is about 4½ in. horizontally and 2¼ in. vertically. With a limited cross-section of the rating flume such an obstruction creates changes from the normal flow condition which affects meter action beyond the allowable limit. Flow conditions become turbulent as the stream filaments come in contact with the meter (which is a flow obstruction) even though the natural flow before submergence of the meter may be ideal.

Turbulence as the result of the presence of the meter in the flow line is objectionable but unavoidable. It is particularly objectionable because its effect on meter rotation is difficult to determine. This factor is, however, subject to some control and, in fact, offers the greatest field for improvement in meters. Improvement may be accomplished by designing a meter rotor so that it engages automatically for its rotation the forward component of the approaching stream forces. An efficient meter must be sensitive; but sensitiveness may be of two kinds—minimum friction and responsiveness to changes in flow direction. In some vertical-axis meters the friction is almost negligible, and yet they are not affected by changes in direction of flow. Such meters appear more conservative when exposed to turbulent flows than actual conditions justify. Although the friction may be slightly greater in horizontal-axis meters, they respond to changes in flow direction so readily as to appear erratic, thus making it impossible to obtain consistent meter indications when the flow is unfavorable to meter work. In neither case does the meter record true flow velocity. The means of testing these features of a meter as described in the paper are unique, but of little practical value. A meter should be as frictionless as possible in order to increase its sensitiveness. As a rule the sensitiveness of a meter is indicated by the lowest velocity at which it starts rotation. Such starting rotation, however (when the ratio between friction and flow velocity is very high), may be caused as much through turbulent forces as by true flow velocity, since these turbulent forces may be the result of the meter's presence in the stream.

The influence of friction in a meter is confined to low velocities and the low velocity curve for a highly sensitive meter is very short. Sensitiveness increases the range of velocities over which the meter is reliable.

Meters cannot be satisfactorily compared unless the tests are made under the most favorable conditions. Ratings made under these conditions bring out information on the efficiency and reliability of a meter. Meters exhibit their characteristics at their low velocity ratings. Accuracy is recognized by the regular distribution of rating points; efficiency, by the shortness of the low velocity curve; and sensitiveness, by the steepness of the slope of the rating curve.

With ideal flow and operating conditions, any meter will act reliably, but to select the proper meter for field conditions one should examine the rating curve carefully. A meter for general purposes, or one which will be required to operate under varying stream or channel conditions, should be of a design having the least flow obstuction; it should have a rotor that will engage the maximum of stream filaments consistent with its size; it should be immune from vertical forces so that it may be used in vertical integration; and, it should have some means of securing a stable position automatically.

Under favorable conditions the records from all meters will agree, but under unfavorable conditions they will differ. This disagreement, however, is no greater than that often obtained from repeated measurements by the same meter. The meter that produces the least interference to flow will produce the most nearly correct readings. To operate properly, it must be held rigidly in such a position in the stream, that it is least obstructing to flow, regardless of the velocity. The meter that presents the least resistance to the flow is most easily held in this position.

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PAPERS AND DISCUSSIONS

RAINFALL CHARACTERISTICS AND THEIR RELATION TO SOILS AND RUN-OFF

Discussion*

By Messrs. George S. Bliss, Brehon B. Somervell, Jesse W. Shuman, and Clinton L. Bogert.

George S. Bliss,† Esq. (by letter).‡—The outstanding features of interest in this paper are the ten columns of percentage values in the tabulated matter and the discussion of precipitation patterns.§ The percentage columns shed new light on some old problems by placing them in new settings.

In the discussion of precipitation patterns a little more prominence might have been given to the part played by topography, which seems to be one of the major factors. Without more or less rugged topography the other factors lose much of their effectiveness. For example, with a topographic map of Pennsylvania, it is entirely feasible for one versed in the meteorology of this section to prepare a fairly accurate outline of the rainfall distribution and to estimate closely the relative differences between the wettest and dryest regions of the State without the aid of precipitation data. The author's deductions from generally accepted meteorological facts and theories seem good.

Steep barometric gradients are not always accompanied by steep temperature gradients. In the ordinary cyclonic disturbances it seems to require a combination of both conditions to produce maximum wind velocities. In instances where the temperature gradient is comparatively flat, the wind velocities are usually moderate despite fairly steep pressure gradients. The greatest wind velocities known are in tornado funnels, which are solely the result of great differences in temperature of adjacent air masses. The extremely steep barometric gradients in the funnels are results of, rather than causes of, the violent air circulations.

If there is any relation between variations in precipitation and sunspot numbers it is completely overshadowed in localities, or over small areas, by accidental causes. It could be detected only by computing mean monthly and

* This discussion (of the paper by C. S. Jarvis, M. Am. Soc. C. E., published in January, 1930, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Senior Meteorologist, U. S. Weather Bureau, Philadelphia, Pa.

‡ Received by the Secretary, January 25, 1930.

§ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 20.

annual rainfall over vast areas, such as a continent or an entire hemisphere. Variations in solar radiation, if there be such, are completely absorbed in the general circulation of the atmosphere, and are not in evidence in the temperature records of any particular region.

This paper as a whole seems to apply more particularly to the problems of flood forecasting, and less directly to those of flood control. Just to the extent that calculations for the latter are based on such stern realities as river-gauge heights and stream-discharge measurements, the variable factors leading up to that stage are eliminated, and accurately so.

BREHON B. SOMERVELL,* M. AM. SOC. C. E. (by letter).†—The author has assembled a mass of basic data which should be of value for the general study of the factors responsible for rainfall and the corresponding run-off. Covering as they do the entire globe, these data should be helpful in analyzing any particular water-shed by comparison with other water-sheds.

Neither the data presented nor other generalities will suffice for a thorough study of a particular water-shed. If stream-flow records are lacking, considerable judgment must be used in drawing conclusions as to the resulting run-off because there are a number of factors affecting run-off, and also because there are different methods for combining these factors.

The factors themselves are fairly definitely known, and the effect of a definite factor, or a definite combination of factors, on the run-off to be expected from a definite amount of precipitation, can be predicted without too great an error. No method is known, however, whereby engineers can predict which combination of factors is going to occur at a specific time, and, consequently, estimates as to run-off will always be subject to considerable error for water-sheds of any appreciable size.

In studying an individual stream, the water-power resources, its navigable capacity, or the menace of its floods, the engineer is concerned particularly with the actual past performance of the stream. A mean of past performances is not sufficient, nor is the record from only one station of great value. Storage must be provided for low-water periods, during which the stream will not supply sufficient water for power or navigation, and storage or other means of flood control must be provided for the high-water period also when valley storage is not sufficient. For these purposes monthly quantities of flow are the smallest increments that may be used safely. If there is no record of these flows, so that they must be computed from rainfall data, then the records from all stations within the area under investigation should be included. For example, in the study of the Potomac River as a source of power the following observations are noted:

Mean Annual Precipitation, Potomac River Basin:	Inches.
From U. S. Engineer Office, approximately..........	41
From Table 1‡ (Appendix I), (Station 75, Column (13)), Washington, D. C..........................	40.5
From Table 1 (Appendix I), (Station 76, Column (13)), Washington, D. C..........................	42.2

* Maj., Corps of Engrs., U. S. A.; Dist. Engr., Washington, D. C.

† Received by the Secretary, February 26, 1930.

‡ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, pp. 32 *et seq.*

Maximum Annual Precipitation Recorded in the Potomac River Basin:

	Inches.
Bayard, W. Va., 1926	89.01

Maximum Annual Precipitation Recorded for Potomac River Basin in Table 1 (Appendix I):

Washington, D. C. (Station 75, Column (14))	61.3
Washington, D. C. (Station 76, Column (14))	61.3
Elkins, W. Va. (Station 74, Column (14))	65.4

Minimum Annual Precipitation Recorded in Potomac River Basin:

Western Port, Md., 1895	12.71

Minimum Annual Precipitation from Table 1 (Appendix I):

Washington, D. C. (Station 75, Column (12))	18.8
Washington, D. C. (Station 72 (Column (12))	30.8
Elkins, W. Va. (Station 74, Column (12))	38.8

Maximum, minimum, and average tabulations at single stations are not conclusive as to conditions even in a relatively small surrounding area, and cannot be used with accuracy in the investigation of a near-by drainage area for water-power potentialities or flood-control studies.

As an instance the drainage basin of the Potomac River is covered in Table 1 (Appendix I) by the stations at Washington, D. C., and Elkins, W. Va. The maximum annual precipitation is recorded at Elkins, as 65.4 in. Actually, the maximum annual precipitation recorded within the Potomac River drainage basin was 89.01 in. at Bayard, W. Va., in 1926, a difference of approximately 24 in.

The same situation occurs in the minimum annual precipitation recorded as 18.8 in. at Washington, D. C., in Table 1 (Appendix I). Western Port, Md., however, in 1895, recorded an annual precipitation of 12.71 in., 6 in. less than the minimum recorded in Table 1 (Appendix I). An entirely different picture may thus be presented, depending upon the stations selected.

From these remarks it will be noted that although the mean for the water-shed derived from a large number of stations approximates fairly closely that given by Mr. Jarvis in his tables for the station at Washington, his maxima and minima are far from those which have been recorded at other stations within the past and also far from the maxima and minima applying to the basin as a whole.

In the study of large water-sheds involving several hundred thousand square miles, the data given in Table 1 (Appendix I) may furnish means, maxima, and minima closely approximating those which might be obtained from a much larger list of stations within the same area. In no case, however, is it believed that one station could be used with any degree of accuracy in a design of works for flood control, power, and navigation.

In other words, comprehensive summaries, such as the author has presented, aid one in studying the local problem by quickly giving one a broader

geographical picture of the rainfall phenomena. They may keep one from drawing false general conclusions from limited local data and may lead the way to additional pertinent information. They do fall short, however, of giving those data essential to reliable conclusions as to the solution of a particular problem.

JESSE W. SHUMAN,* M. AM. SOC. C. E. (by letter).†—This paper is a valuable contribution to the subject, with the points well brought out and the conclusions carefully worded. Undoubtedly, the views expressed are a résumé of the current opinions of both meteorologists and hydrologists. The attitude of the former is reflected in the statement,‡ "no direct relation is traceable between rainfall and sunspot periodicity * * *"; and the attitude of the latter is indicated by the methods suggested for study of the rainfall data by means of frequency curves, the consideration of floods, etc.

Meteorologists are quite aware of the cycles in weather phenomena, but, in general, they make no application of this knowledge, probably because the day-to-day or month-to-month variation in any of the generally used meteorological data is negligible. Occasionally, papers appear in the *Monthly Weather Review* that deal with this subject authoritatively, from the viewpoint of meteorologists and mathematicians as well as "dyed in the wool" hydrologists. Stripped of all their involved mathematics, meteorological terms and background, the crux of these papers is, that by proper treatment of sunspot numbers and rainfall or run-off data, a relationship does exist, which is easily discernible. The papers also show that, while these cyclic variations are negligible in their effect from day to day as to current weather changes, nevertheless, their effect is cumulative and works great changes in run-off, lake levels, etc.

The writer does not hereby intend to decry the work of the author; rather he wishes to show in what way the valuable data assembled by Mr. Jarvis can be further considered and used to great profit.

The author's Fig. 2§ compares a graph of sunspot numbers with graphs of several rainfall stations. From an inspection of the graphs, Mr. Jarvis draws the conclusion that there is no traceable relation between the sunspots and the rainfall. The method of comparison here used is perfectly standard in its application, and it is readily admitted that, with the graphs as drawn and located, there is no easily recognizable relation.

In Fig. 8, the same data for sunspot numbers are used,‖ and graphs of rainfall at Milan, Italy, New Orleans, La., Charleston, S. C., and Greenwich, England, are shown. In Fig. 2 the sunspot numbers are inverted, with zero numbers at the top; whereas Fig. 8 shows the zero numbers at the bottom. Furthermore, Fig. 8 has a much larger vertical scale. After plotting all the sunspot numbers (the mean Wolf and Wolfer provisional number for the year) for the entire record, a smooth line is drawn through the plotted points. This yields the series of loops shown. This curve, thus far, is identical with Fig. 2,

* Secy-Treas., Power Eng. Co.; Cons. Engr., Minneapolis, Minn.

† Received by the Secretary, March 10, 1930.

‡ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 22.

§ *Loc. cit.*, p. 23.

‖ From *World Weather Review* and *Monthly Weather Review*.

except that it is inverted. Each peak or crest represents maximum sunspot numbers, and the troughs represent minimum sunspot numbers. When viewed in this state, there is no apparent relation with rainfall. However, the curve may be further reduced. Divide the distance between each peak and trough (as at 1 and 2) with a dot; then pass a smooth line, Curve *B*, through the dots. This curve is the well-known Bruckner cycle; it is merely the median line traced through the smoothed sunspot numbers, which is the same line one would obtain if the sunspot numbers graph were repeatedly integrated. The peaks of this cycle represent, of course, relatively higher sunspot numbers. They occurred in 1787, 1803, 1837, 1870, 1893, and 1917.

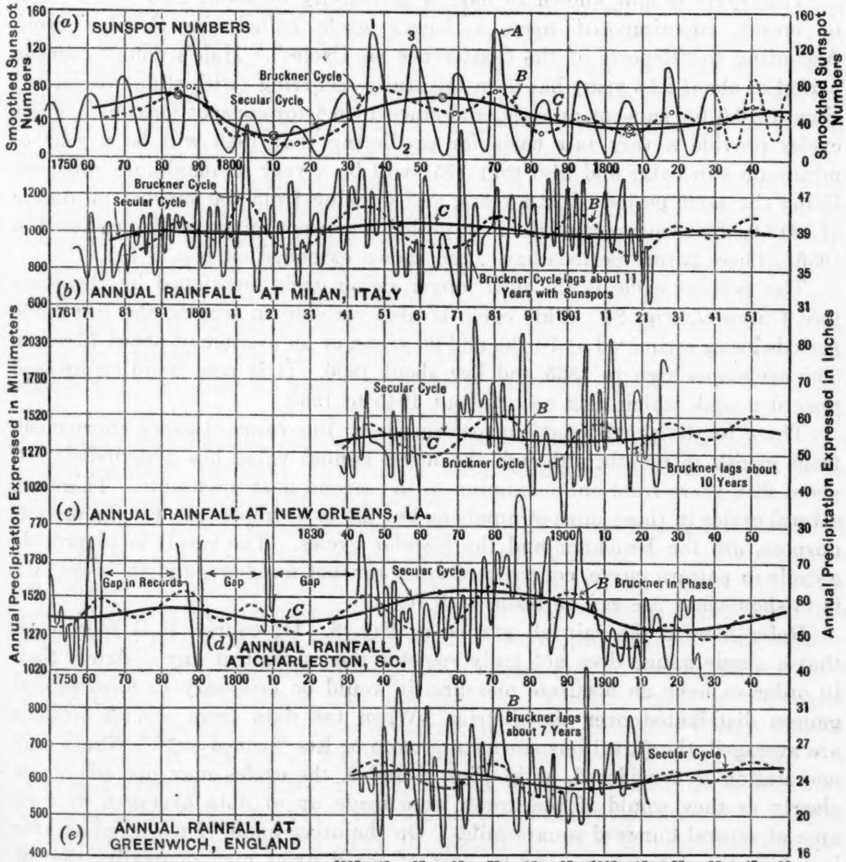

FIG. 8.—COMPARISON OF BRUCKNER CYCLES FOR SUNSPOT NUMBERS WITH CYCLES OF MAXIMUM RAINFALL.

Bruckner's discovery of this cycle was accomplished from studies* (mostly of European data) of secular variation of the Caspian Sea; lakes and seas without outlet; river heights; precipitation; atmospheric pressure; temperature; and times of grape harvest.

* "Klimaschwankungen seit 1700," 1890, Vienna.

Although suspecting a relation between solar variation and terrestrial phenomena, Bruckner's efforts in this direction were unsuccessful. He simply reduced his statistical data by the use of five-year running means, and this method is insufficient to segregate the cycles.

Meteorologists discarded the cycle because it gave no reliable clue as to present-day variation in weather; and, further, because there was apparently no connection between it and any great "cause". With the advent, however, of the publications of the U. S. Weather Bureau, methods of analysis, with examples and illustrations were given, showing more clearly the nature of the Bruckner cycle.

This cycle is now known to have a periodicity of about 22.6 years (crest to crest), superimposed upon a longer cycle called the Secular cycle. According to "Reports of the Conference on Cycles",* Hale's double sunspot period of about 22.5 years has been faithfully recurring (with slight variation) since 1810.6 for sunspot minima, and since 1816.4 for sunspot maxima. These cycles provide a very fair basis for predicting that 1946 will be a year of minimum sunspots; and also that 1951 will be a year of maximum sunspots. Using the same period of 22.6 years, and working from the sunspot minimum of 1913.6, the occurrence of the next probable sunspot minimum is predicted in 1936. These future periods have been shown extended on Fig. 8 (*a*).

The Secular cycle is the next longer swing cycle mentioned by Bruckner (see Curve *C*, Fig. 8). This cycle is also variable in length, the last three periods being estimated at 70, 60, and 90 years, or an average of about 73 years. The cycle was high in 1855 and low about 1906. It is now trending upward toward a peak which may occur about 1945 to 1950.

Thus, in the provisional sunspot numbers, the value changes enormously from month to month, although the mean annual value has a periodicity of about 22.6 years from one maximum to the second next maximum. There are several cycles in these sunspot numbers, but the most important, for the present purpose, are the Bruckner and the Secular cycles. The result is to provide a cycle or pattern curve, constructed from sunspot numbers, and the next task is to show what use can be made with it.

Refer now to the rainfall graphs in Fig. 8. Of course, it is recognized that a single gauge does not truly register the rainfall at any district; that, in order to have an accurate measure, it would be necessary to have several gauges distributed over the district. When the data from several stations are averaged, the fortuitous storms are more or less "ironed out". Where only one station is considered, as in Figs. 2 and 8, the cycles may not emerge as clearly as they would if the graph were made up of data averaged over an area of several hundred square miles. On the other hand, if too large an area is taken, the data at one end or part of the district may neutralize that of another section, and one may secure no recognizable cycles. Preferably, then, eight or ten Weather Bureau rainfall stations should be included, and their data should be averaged. After one has gained experience in this type of analysis, it is possible to trace in the cycles in the data of a single rainfall station, as is done in Fig. 8.

* Carnegie Institution of Washington, 1929.

Consider the graph of Milan rainfall (Fig. 8(*b*)). Here is a long record that, by proper treatment of its data, can be made to show both the Bruckner and Secular cycles. The treatment consists, essentially, of smoothing processes, and separation of the original graph into its component parts.

Assume, therefore, that these processes have been gone through with the Milan rainfall data and that, finally, the Bruckner cycle has been derived. Plot this cycle in the original graph, and then compare the graph with that of the sunspot numbers. In order to get the best synchronism between the Bruckner cycles in the two graphs, it will be necessary to set 1881 on the Milan graph on the 1870 ordinate of the sunspot graph. No attention is given to the amplitudes of the cycles; only the synchronism of the peaks and depressions is important. Fig. 8 shows that the trend in rainfall at Milan lags behind that of the sunspot numbers by about eleven years. If one looks at the Milan graph trying not to see any cycles, it seems only like a series of oscillating values, with no law or order. With the cycles drawn in, however, it certainly is obvious that, in general, the trend of wet and dry periods is in accordance with the "ups" and "downs" of the cycles.

In attempting to compare rainfall or run-off graphs with the comparison curve, that is, with the sunspot Bruckner cycle, the data of one cannot be compared with the current data of the other, there may be a lag or a lead; only occasionally are both sets of data in phase with each other. Bruckner thought that the phase of his cycle was the same all over the world, but now it is known that the phase varies depending on the geographic locality; that is, along the Atlantic Coast, at Boston, Charleston, etc., the trend of rainfall is about in phase with the sunspot numbers. Toward the West, however, the phase begins to lag, until a maximum lag occurs at the Pacific Coast. The phase may be different even within the confines of a single State. The prime requisite, therefore, in making comparisons of rainfall graphs is to determine first the phase of their trends.

The Secular cycle in the Milan graph is high in 1855 and low in 1906, the same as it is in the sunspot graph, because the phase of this cycle seems to be the same all over the world. Fig. 8(*b*) shows where the highest values for total annual rainfall occurred; that is, from 1841 to 1851, which is at the crest of the Bruckner, and near the crest of the Secular cycles. One expects higher values at or near the crests of these cycles, and, of course, lower values at the troughs. The graph indicates that the Bruckner cycle is the major control (because such low values occurred due to the Bruckner trough in 1861 to 1871, even if this period was very near to crest of the Secular cycle). A Bruckner crest is indicated in 1930, so that moisture conditions should now be above normal; and the next great wet period is indicated about 1945 to 1955.

In Fig. 8(*c*) the Secular cycle of rainfall in New Orleans is quite flat, with small amplitude. The Bruckner cycle is responsible for the great swings in moisture conditions in this case. Note that 1880 of the New Orleans scale is set on the 1870-year ordinate of the sunspot graph, thus indicating a lag of about ten years at that point.

In Fig. 8(*d*) the Bruckner cycle for rainfall at Charleston, S. C., is in phase with the sunspots trend, as indicated. The maximum wet period of the

record occurred after the crest of the Secular cycle. The graph shows a trough of the Bruckner cycle at 1929, indicating a control toward lower rainfall values. Since there has been such a great drop from 1924 to 1927, the rainfall for 1928 is more than likely to be quite high. A study of rainfall records shows that Nature tends to preserve its balance, as it were; and that extreme values are likely to be followed by extreme values of opposite sign the following year or period.

Finally, Fig. 8(*e*) is for the rainfall at Greenwich, England; the lag is about seven years. The Bruckner cycle was at crest in 1927, indicating moisture conditions above normal. These few examples should suffice to show that:

(1) Rainfall has a distinct, traceable relationship with sunspots.
(2) Proper attention must be given to choice of vertical and horizontal scales used for plotting; otherwise, the cycles, whose amplitudes of swing may be very slight, will not easily be discernible.
(3) All rainfall graphs follow the same general pattern, even if they may differ materially in the limits of their departure from the mean value.
(4) After the phase of the trends of rainfall has been determined, with respect to the long swing curve in the sunspot numbers, a great similarity between the different rainfall graphs is observed. Rainfall all over the world is brought about by the same "causes", even though the lapse of time before the "effect" takes place, varies.

While the foregoing remarks have been applied to rainfall they may cover run-off data as well. In fact, run-off data give much better cycles, as the drainage areas act as integrators. The advantage of being able to identify these cycles, is simply that they provide an index of the trends of the graphs, which is of value for bridging gaps in the past, and for indicating the probable times of future peaks or troughs, some years in advance. The extreme of rainfall, as listed in Table 1 (Appendix I),* can now be used to greater advantage, especially if the year and month of their occurrence is noted.

If a hydrologist knows when to look for a peak in the Bruckner cycle in rainfall data at a certain locality, he has a good idea of the period of flood occurrence, referring to the great floods. Floods do occur at or near the troughs of the Bruckner curve, but they do not reach the magnitude of the others. Each section of the country needs to be studied individually. The times of flood occurrence can be marked on the Bruckner cycle in the rainfall graph, and note can be made of the position of such occurrence. It is believed that greater accuracy as to flood occurrence and magnitude can be secured by this type of study than by treating all flood data by means of frequency curves and probability methods.

A. Streiff, M. Am. Soc. C. E., has shown how to estimate the total run-off on a river for future years if twelve years of monthly means, determined by fairly good gauging, are available.† Considering also that most rivers lag behind the rainfall over their drainage areas from one to five months, it is quite obvious that hydrologists need no longer remain in the dark as to the

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 32 *et seq.*
† *Monthly Weather Review*, March, 1928.

total annual water output of the river for the next year. The extremes of discharge during the year, constituting floods, require much further study.

The fact of lag or lead in phase of the trends of rainfall, with the common great cause, explains why local floods occurred in Alabama in the autumn of 1929, while at the same time the region around Seattle, Wash., was suffering from one of its worst droughts. Each section of the country, part of a State, etc., is performing its cycle of operations, giving now wetter and now drier weather, in accordance with these cycles. If the troughs or crests of the Bruckner and the Secular cycles happen to come close to phase, the extremes of rainfall occur. If the parts of the country which feed rivers that eventually come together (such as the drainage areas of the Mississippi River) happen to have cycle phases that practically coincide, then the great floods such as that in 1927, occur.

The graph of Greenwich, indicates a peak of the Bruckner cyele about 1929 (Fig. 8(*e*)). On January 7, 1928, one of London's worst storms occurred. The graph of New Orleans, La. (Fig. 8(*c*)), indicates a Bruckner cycle crest about 1926. On September 8, 1929, this city had a great storm, with 10.75 in. of rain falling in 24 hours.

It is conceivable, then, that the great floods of the Mississippi are the results of the phase coincidence of these important cycles. The writer suggests that the Mississippi River Commission make an analysis along the lines outlined, studying each tributary river first, then combining the results for a grand total and a grand resulting series of cycles. Noting on these curves the flood times that have occurred in the past, the Commission should have much more data by which to predict future floods than it has by means of previous studies that have all been based on the theory of probability. Probability deals with numbers that occur purely by chance. It is the writer's opinion that rainfall and run-off data are not chance numbers, but are related to a great "cause".

Clinton L. Bogert,* M. Am. Soc. C. E. (by letter).†—Only those who have put in tedious hours, days, and weeks in compiling, classifying, weighting, and averaging long-term rainfall and stream-flow records can really appreciate the immense amount of drudgery which the author has undergone in producing Table 1 (Appendix I).‡ The profession should be deeply obligated to him for the form and extent of the results.

In this table the author has attained his desideratum of rigorous condensation, at a sacrifice of some essential data. The usability of the data in localities where stream-flow records are published, would have been increased materially were the year entered in the record when each minimum and maximum rainfall (both yearly and monthly) occurred. The dates would also be of value in investigations of droughts and amplitude of rainfall cycles.

In using any observed data, such as rainfall, there is one element of error which is often overlooked: All observers are assumed to make and record

* Cons. Engr. (Sanborn & Bogert), New York, N. Y.

† Received by the Secretary, March 27, 1930.

‡ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 32 *et seq.*

their observations with equal conscientiousness and accuracy. Of course, this is humanly impossible. The observations from which Table 1 (Appendix I) is made up, run into the millions, and to all, the same accuracy is assigned. The author's method of expressing results in percentages will reduce this error of observation and recording to some extent.

The value of Table 1 (Appendix I) for further research would have been enhanced had there been space available for assigning to each of the 820 stations a specific reference as to the source of the data.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

PLASTIC FLOW IN CONCRETE ARCHES

Discussion*

By Messrs. Edward Godfrey, J. H. Griffith, and Charles W. Comstock.

Edward Godfrey,† M. Am. Soc. C. E. (by letter).‡—This paper is valuable, not so much because it proposes a new theory for the solution of the arch problem, but because it demonstrates the inapplicability of all elastic theories to concrete arches as well as the practical impossibility of fitting a workable theory to the many sided problem of the concrete arch that will fit accurately into every condition.

For many years the writer has held that the elastic theory is not applicable to concrete or masonry arches.§ The reason this theory is not applicable is fundamental. In fact, there are many reasons, and they are all fundamental.

The elastic theory pre-supposes a homogeneous and perfectly elastic material in the arch. Concrete is far from homogeneous and far from being perfectly elastic. It would seem superfluous to point out specifically why the elastic theory as applied to the arch required these conditions, but at the expense of appearing pedantic they will be emphasized, for this theory has taken a hold on the profession that seems impossible to shake.

Every engineer knows that to satisfy the elastic theory it is not sufficient that a structure shall show merely what is commonly thought of as elastic properties; that is, it is not sufficient that an arch, for example, can be sprung by a load, and when that load is removed, it will return to approximately the original shape. This, in fact, is all that can be said of a concrete arch. It is about all that was proved by the famous Austrian tests cited by the author.|| The amount of the spring that "agrees with theory", could be made to agree with many other theories by merely assuming or working out a modulus of elasticity to fit the theory.

* Discussion on the paper by Lorenz G. Straub, Jun. Am. Soc. C. E., continued from April, 1930, *Proceedings*.

† Structural Engr., Pittsburgh, Pa.

‡ Received by the Secretary, February 14, 1930.

§ "Concrete" by Edward Godfrey, M. Am. Soc. C. E., 1908, p. 14; *Transactions*, Am. Soc. C. E., Vol. LXX (1910), pp. 64–65.

|| *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 59.

The average modulus of elasticity as found by specimen tests, in the Austrian series of tests on blocks, was about 2 700 000 lb. The "practical" value as determined from analysis of a plain concrete arch was 1 430 000 lb.* or nearly 100% difference. There were no fixed-ended arches among them. There was a long plain concrete arch and a long Monier arch. These are the tests which the Engineering Profession points to when the elastic theory is questioned as to its applicability to reinforced concrete arches and on the basis of which fixed-ended arches are confidently designed. Because a plain concrete bow between solid abutments deflected in an elastic curve, reinforced concrete arches between settling abutments are designed with fixed ends.

A uniform load on an arch, and an agreement with a theory in the observed measurements under such load, are far from being sound arguments for the extension of that theory to include unsymmetrical loading. Every engineer knows that a material to be elastic in the meaning of the theory, must be first homogeneous. Some of the Austrian tests were on brick and stone masonry. By no stretch of terms can these be considered homogeneous; hence, agreement with theory means nothing, if that theory is an elastic theory.

Concrete is not homogeneous. The properties of the stones are not the same as those of the mortar. One batch of concrete will differ from another. Interior concrete will differ from exterior concrete. Shrinkage will affect the thin parts more than the thick parts. The unstressed shape of the arch can never be known. When the arch is wet, it will expand, altering the stresses; yet the elastic theory must assume entire homogeneity as well as constancy of condition.

To be elastic the concrete must return to its original shape after being stressed; yet concrete takes a permanent set of an unknown amount and of an amount varying with the age and other conditions. It is quite impossible to evaluate these conditions. On this fundamental basis, the elastic theory for arches is eliminated from serious consideration, as Mr. Straub points out.†

Fixed-endedness is a vital part of the elastic theory. This means absolutely rigid abutments; it means no horizontal movement and no rotation. Either of these influences vary, materially, the stresses in the arch. Horizontal movement is to be expected in any arch not abutting against vertical rock faces. Rotation, even in an arch of 119 ft. span founded on rock, was reported in a discussion before the Society.‡

In still another respect concrete fails to meet the mathematical or engineering criterion of an elastic substance. To be elastic in this sense there must be a constant ratio between stress and strain for all intensities of stress within the allowable limits. This is a fundamental assumption of the elastic theory. Concrete is far from meeting this criterion.

All these fundamental bars to the elastic theory as the solution of the concrete arch problem have been known and emphasized for many years. The elastic theory persists, however, and the most elaborate mathematical

* *Engineering News*, April 9, 1896.

† *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 49.

‡ *Loc. cit.*, March, 1924, Society Affairs, p. 292.

computations are carried out on the basis of this theory. There is scarcely any other problem in engineering in which such intricate mathematical processes are utilized. In the face of all the intricacy of the elastic theory and the difficulty of designing an arch by this method it is only to be expected that the advocates should show strong justification for its use and close agreement between theory and fact in the matter of stresses. This, they are unable to do, as repeatedly emphasized by the author.

It will be urged, as it has been, that these objections to the elastic theory are on the high ground of a demand for perfection, that even steel is not perfectly elastic. It is a far cry from the trifling departures from perfect elasticity and perfect fitting of the unstressed steel arch in the position for which it is designed, to the 100% departures from ideal conditions proved to exist in the case of concrete.

There is a psychological hazard in complex theory. The mere application of the theory, the working out of an intricate problem begets a confidence that is not engendered by an approximate method. Factors of safety are cut to finer limits when stress coefficients are worked out to the fifth decimal place (which is not unheard of), based on erroneous premises. When a method of design is frankly set up as an approximation, the designer will naturally take this into account in choosing a factor of safety. The writer has noted factors of safety given in tenths and hundredths (exceeding unity) where the structure that was "safe" had failed. The psychology of engineering may be of as much importance as its mathematics. After all, engineering is not merely the working out of a mathematical problem, no matter how nicely that problem may develop under the solution applied by higher mathematics.

It is of supreme practical importance that designing be simplified where possible. Comparatively few engineers, perhaps less than 1% of the men engaged in designing, could follow the solution of the concrete arch problem by the elastic theory.

One of the principal considerations in the design of an arch is provision in the cross-section of the arch ring for the thrust due to full uniform live load. This is, in fact, the first consideration in point of time. Symmetrical uniform load is not usually the critical condition of loading, however. If full load on an arch were the critical loading, it would matter little what theory were used. Furthermore, it would make little or no difference whether the arch were considered as fixed-ended or hinged-ended. The simplest "row-of-blocks" method would give the same answer as the most complex elastic theory. It is simply a matter of resolving the forces, applied loads, and end reactions, into the proper equilibrium polygon.

The author points out* the fact that his theory and the elastic theory are in practical agreement for the condition of uniform live loading. It is when unsymmetrical loading and provision for such loading are taken up that the several theories diverge. The author points out that when the real properties of concrete are theoretically taken into account—the non-elastic properties, so to speak—it is not possible to determine the distribution and intensity of stress on a concrete cross-section, given the direct compression

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 69.

and the bending moment. It is necessary in the solution, the author shows,* to know the point of application of the force producing the bending moment. This not only complicates the solution of the problem on the author's proposed theory, but makes it practically impossible to determine the distribution of live load that will constitute the critical condition for design. The fact is that the problem of determining the critical position of live load for maximum extreme fiber stress introduces such complications into the elastic theory itself that the question is touched upon but lightly in treatises and papers on that subject.

It is significant that arch tests that are used to sustain the elastic theory are not made on critical loading on the arches. It is such testing that would be useful in checking up the theory and furnishing a basis for confidence in that theory.

The crux of the situation as to concrete arch design centers on the assumption of fixed-endedness, invariably made when the elastic theory is applied. There would be no use whatever for the elastic theory in the design of concrete arches, if the arches were assumed as hinged-ended; and here is another of the anomalies of theoretical engineering. The fixed-endedness of an arch pre-supposes massive, immovable abutments, and yet theoretical treatments of elastic arches gives little or no attention to the proportions of the abutments that are necessary to give the needed rigidity.

It is usually difficult to provide sufficient width, economically, in an abutment to bring the line of thrust within the middle third of the base, assuming the arch to be hinged-ended; and this assumption means a practically fixed line of thrust through the abutment. By the methods of the elastic theory the line of thrust through the abutment for unsymmetrical load, may strike through the upper edge. A large and sometimes prohibitive addition in the mass of the abutment, would be demanded, if this were properly taken into account. The extra concrete required in this case to give the abutment its needed stability far outweighs the saving of a few inches in the thickness of the arch ring which, theoretically, is effected by the methods of the elastic theory and assumed fixed-endedness.

J. H. Griffith,† M. Am. Soc. C. E. (by letter).‡—The writer is impressed with the fundamental character of this paper and by the fact that much thought and careful study have been given to its preparation. Apart from the author's elaboration of his thesis he calls attention from time to time to the disparities existing between the scientific and technologic viewpoints in interpretations of the phenomena of elasticity, plasticity, viscosity, and other properties of matter. Notwithstanding the profound researches upon the subjects by Dr. Eugene Cook Bingham and by Charles Terzaghi, M. Am. Soc. C. E., the author feels the questions at issue are mooted ones. The writer would like to discuss more particularly in this connection some of the divergencies between scientific and technologic opinions with the view to a reconciliation.

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, pp. 67–68.
† Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.
‡ Received by the Secretary, February 17, 1930.

Without a certain unity and purpose little progress can be made in research work. It may be stated that the writer's recent research has been limited largely to studies of the phenomena of earth resistance. In this case, however, the anomalous behaviors encountered in studies of the phenomena of plasticity and viscosity are rather more pronounced than those which usually occur in the instances of concrete, freshly quarried rock, or green timber.

Maxwell's definitions upon the fundamental properties of matter under consideration are pertinent to the discussion; he remarks:*

"Suppose that stresses of the same kind, but of continuously increasing magnitude, are applied to a body in succession, as long as it returns to its original form when the stress is removed, it is said to be perfectly elastic. If the form of the body is found to be permanently altered when the stress exceeds a certain value, the body is said to be soft or plastic, and the state of the body when the alteration is just going to take place is called the limit of perfect elasticity. If the stress be increased till the body breaks, or gives way altogether, the value of the stress is called the strength of the body for that kind of stress. If breaking takes place before there is any permanent alteration of form, the body is said to be brittle. If the stress when it is maintained constant causes a strain or a displacement in the body which increases continually with the time, the substance is said to be viscous. When this continuous alteration of form is only produced by stresses exceeding a certain value, the substance is called a solid, however soft it may be. When the very smallest stress if continued long enough will cause a constantly increasing change of form the body must be regarded as a viscous fluid, however hard it may be. Thus, a tallow candle is much softer than a stick of sealing wax: but if the candle and the stick of sealing wax are laid between two horizontal supports, the sealing wax will in a few weeks of summer bend with its own weight while the candle remains straight. The candle is therefore a soft solid, and the sealing wax is a viscous fluid."

From the critical viewpoint, it is natural to ask: "Is concrete a viscous fluid or a plastic solid?" According to the author's intimation concrete has no sensible elastic limit except it be that at the origin of the stress-strain curve, these curves being usually drawn continuously concave toward the axis, which is commonly taken horizontal. If concrete is a plastic solid as implied in the author's title, then according to Maxwell's definition, as mentioned, it should have a sensible elastic limit differing from that at the origin, which, however, the standard treatises do not show. This fact is tacitly admitted by the author in his citation relating to Bach's researches.†

The writer has found in his own researches that if the test load is applied slowly enough on cylinders of earth taken from their natural beds, the stress-strain curves from the data are similar to those of concrete and cast iron in that they are continuously curved toward the strain axis. When the stress-strain readings, however, are taken quickly, but without appreciable impact, say, in the space of 15 to 30 sec., there is a sensible elastic limit, with the stress-strain curves ensuing in a manner quite similar to those found for steel columns.‡ On the other hand, in the case of steel columns, when considerable refinement is used in making the strain measurements, there is not, strictly

* "Theory of Heat", p. 295.

† *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 70.

‡ See *Technologic Paper No. 101*, U. S. Bureau of Standards, for curves.

speaking, a perfectly constant modulus of elasticity, nor a true elastic limit for steel. The existence of these functions is more or less a relative matter, and is conditioned to a large extent by the time-rate of application of the load. One of the great educational advantages in observing earths, apart from the immediate technologic needs, is that, being a "visco-elastic" material, the behavior of earth shows in a magnified way what may be expected to occur in other apparent matters so that an investigator can often anticipate phenomena and properly interpret them dynamically. A perfectly elastic matter, so-called, gives a specific and limited rather than a general behavior exhibiting all the properties of matter. A good illustration of that is afforded in the instance of heterotropic crystals compared with isotropic or amorphous matter, such as many soils.

The recent researches by Dr. Abram F. Joffé, of Leningrad,* a collaborator of Röntgen, have a direct bearing upon most of the points under discussion. His researches upon atomic and crystal structure substantiate the opinions which have long been held as to the approximate nature of Hooke's law of proportionality of stress to strain. He sustains the judgment of George Green,† with Cauchy, one of the recognized founders of the theory of elasticity, and of Boussinesq, who conceived the importance of developing stress functions in series as considered later. Boussinesq‡ developed many functions in series. He regarded ordinary matter an intermediate type between the so-called perfect fluid and perfect solid. He so arranged his functions§ that this intermediate matter would merge with the perfectly continuous matter of the theory of elasticity as an extreme case. Dr. Joffé gives many interesting statements bearing upon Hooke's law and the so-called elastic after-effect. In one instance he was in some doubt as to whether the particular type of crystal could be considered as a solid. His remarks on the relation of the elastic after-effect to hysteresis should be considered in connection with the author's interpretation of the phenomenon.

Many of Dr. Joffé's observed phenomena studied by means of the X-ray spectrometer and electrical processes may be observed upon the macro-structures as well. It is known from the strain measurements conducted at the Washington Monument by the late Dr. Becker and Dr. Van Orstrand‖ that steel is in a continual state of flux, the observations being conditioned by the intensity of loading, its rates of application, and the degrees of magnifications of the different strains. It is also a fact of experience that both green and seasoned timber flows continually an infinitesimal amount with the time. On account of this it is necessary for the U. S. Forest Service to adopt certain rates of deformation in order that test data on timber may be properly interpreted and compensated. It is shown by Withey and Aston¶ that a maximum strength of 100% on long leaf yellow pine timber, under momentary or short-

* The Physics of Crystals", 1928, pp. 23, 33, and 44.

† "The Mathematical Theory of Elasticity", A. E. H. Love, Fourth Edition, p. 11.

‡ "Essai Theoerique sur L'Equilibre des Massifs Pulvérulents", par M. J. Boussinesq, Bruxelles, 1876, pp. 6 *et seq.*

§ *Loc. cit.*, Chapters I, II, and III.

‖ Unpublished Memoir, kindly loaned the writer by the Acting Director of the U. S. Geological Survey.

¶ Johnson's "Materials of Construction," Fifth Edition, p. 207.

time loading, will be reduced to only about five-eighths of the value with a continuous application of the loading for a period of 700 hours.

The writer has observed in his own researches that earths cut from the natural beds flow continually in almost imperceptible, but easily measurable, amounts. This flow continues for many days without reaching statical stability when the viscous sub-matrix or "binder" ceases to function from a complete "grain gearing" under the applied load. When there is complete particle gearing, twelve particles are in contact around a particle nucleus, as in the case of annealed glass, and in conformity with the structures of granular matter discussed by Dean C. S. Slichter* and the late Professor Osborne Reynolds† for the different orders of matter considered in their respective fields of investigation. In the case of experimental piles sunk into the earth, there is a tendency for the pile to resume the unstrained position after the load is removed if time is given. It is also well known that when proof loads are applied to steel and cast-iron members well within the elastic limit the stress-strain curves tend to straighten conformably to Joffé's observations and presumably owing to the fact that the crystals of the metal tend to approach the configuration of maximum density already cited.

If matter flows very slowly, but still perceptibly, like a glacier, with the phenomenon of flow occurring from the beginning of the time of stressing, it would seem appropriate, in the writer's opinion, that the behavior should be called viscous instead of a plastic phenomenon in accordance with the scientific nomenclature. It is questionable whether this flow should be regarded as hysteresis according to the author's point of view, which is the lag experienced in the case of cyclic stress and similar phenomena in other fields of science. The supposition of a truly reversible process in any of Nature's laws is, of course, a purely analytical fiction made for convenience mathematically. One may accept it as an induction of experience that there is always some degree of dissipation of energy. In the extreme case of light transmission, Professor Osborne Reynolds reached the conclusion that the viscous dissipation is sufficient to reduce the total wave energy to one-eighth in 56 000 000 years.‡

The phenomenon of a flux of matter disappears entirely when there is an absence of the viscous sub-matrix in the particle interspaces which possesses the nature of a grain lubricant. This occurs in the case of dry sand and gravel under pressure partly from the particles getting on dead centers, as in the case sometimes occurring for locomotive drivers, and partly from the building up of frictions under loading. According to Boussinesq the particles gain an elasticity of bulk. The writer demonstrated this by loading tracing cloth cylinders, 4 in. in diameter by 14 in. high, filled with gravel and tested to destruction. The cylinders became as hard as flint as the grain frictions built up under loads of 800 to 900 lb.,§ thus affording a good hypothesis as to why tool steels harden when heated above the critical range of temperature and are quenched in oil or water. The hoop tension supplied

* 19th Annual Rept., U. S. Geological Survey, Pt. II, p. 306 *et seq.*

† Collected Works, Vol. III, p. 1 *et seq.*

‡ *Loc. cit.*, p. 238, Article 258.

§ *Proceedings*, Am. Soc. C. E., August, 1920, Papers and Discussions, p. 934.

by the outer shell of contracting material causes a gain in bulk elasticity from the up-build of pressure and inter-crystal friction.*

Inasmuch as the author's criticisms of the theory of elasticity evidently relate to the theory as used by technologists, and there is often a wide discrepancy here between that and the classical theory developed by scientists and engineers, the writer will discuss some of the fundamental principles of the rigorous mathematical theory to correct some of the impressions commonly held. Hooke's law, as the author has shown,† should be properly interpreted. This law should be regarded a limiting aspect. It strictly applies solely to infinitesimal strains; it does not take cognizance of finite strains as sometimes used by technologists, except to a first and sometimes a very rough approximation as the author remarks in effect. George Green,‡ in formulating his dynamics of potential, conceived that the functions could be developed in power series. The idea is clearly set forth by William H. Burr, M. Am. Soc. C. E., who remarked more than forty years ago§:

"It is a matter of experience that strains always vary continuously and in the same direction with the corresponding stresses. Consequently, the stresses are continuously increasing functions of the strain, and any stress may be represented by a series composed of the ascending powers (commencing with the first) of the strains multiplied by proper coefficients. When, as is usually the case, the displacements are very small, the terms of the series whose indices are greater than unity are exceeding small compared with the first term, whose index is unity. These terms may consequently be omitted without essentially changing the value of the expression. Hence, follows what is ordinarily termed * * * Hooke's Law."

Since there is but one fundamental method of analytical continuation known to mathematicians, the various methods used being equivalent to Taylor's, the French mathematician, Boussinesq, proceeded along strictly logical analytical lines in his development of functions by Maclaurin's well-known formula, this being a special case of Taylor's formula, in handling the finite displacements of granular materials. A good illustration of such a development is illustrated in Love's development of strains by Taylor's theorem of continuation. For large strains he obtains expressions of the types‖:

$$(\delta_{xx}) = \frac{\partial \mu}{\partial x} + \frac{1}{2}\left[\left(\frac{\partial \mu}{\partial x}\right)^2 + \left(\frac{\partial v}{\partial x}\right)^2 + \left(\frac{\partial w}{\partial x}\right)^2\right]$$

$$(g_{yz}) = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z} + \frac{\partial \mu}{\partial y}\frac{\partial \mu}{\partial z} + \frac{\partial v}{\partial y}\frac{\partial v}{\partial z} + \frac{\partial w}{\partial y}\frac{\partial w}{\partial z}$$

for finite normal and shearing stresses. When the strains are ultimately small in accordance with Professor Burr's description, the higher order derivatives vanish, as he states, leaving the ordinary expressions used by engineers and elasticians, or,

$$\delta_{xx} = \frac{\partial \mu}{\partial x}$$

$$g_{yz} = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}$$

* Johnson's "Materials of Construction", Fifth Edition, p. 634.

† *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 58.

‡ "The Mathematical Theory of Elasticity", A. E. H. Love, Fourth Edition, pp. 11-12.

§ "The Elasticity and Resistance of Materials", Second Edition, p. 2.

‖ *Loc. cit.*, p. 60.

Since the author* raises the question of the propriety of the usage of initial, tangent, and secant moduli, the writer also calls attention to the fact that Professor Burr gives the correct analytical expression of a modulus as the derivative of stress with respect to strain. This may be constant for a plastic material below the elastic limit; it is a variable for viscous materials generally and for plastic materials above the elastic limit. From the critical standpoint the modulus must also be defined with respect to the state of matter existing. For example, the late Professor William Kent when discussing thermo-dynamic relations in his well-known "Mechanical Engineers' Handbook" treats the subject of moduli at constant pressures and at constant volumes; but since a scientist must deal with a number of variants in defining the states of matter in conformity with such well-known laws as Willard Gibbs' phase relation, van der Waal's, Berthelot's, Keyes', and the hundred others proposed,† it is correct to speak of moduli at constant temperatures, at constant density, or in terms of chemical composition, three states usually being sufficient. In the theory of earth resistance the pressure and density are the important variants. In the correct usages of the theory of elasticity as distinguished from the many nascent soil theories being proposed it is necessary to use a number of variants; pressure, volume, temperature, density, and chemical variants arise as instanced in the case of simultaneous water, ice, and steam phases. Kelvin and Kirchhoff were particularly concerned with isothermal and adiabatic straining in defining the existence of a potential function.‡

The author's exponental functions for stress-strain relations and time-yields are of the nature of pioneer work; such relations have the support of C. Bach and other German authorities. G. S. Williams, M. Am. Soc. C. E., and A. N. Talbot, Past President, Am. Soc. C. E., have made considerable use of expressions of this kind notwithstanding their empirical forms. There is a deviation here to be noticed between the author's types of empirical equations and those of the rigorous theory of elasticity. The rigorous expressions are the well-known equations of motion (or rest in particular) of the theory of elasticity.§ They are analytically equivalent to Newton's second law, force = mass times acceleration, resolved along the particular co-ordinate axes chosen. The labor involved whether by Mr. Straub's analyses or those of the theory of elasticity is relatively enormous. The plan used extensively by the writer in laboratory measurements has been to make direct correlations of the experimental data with those of the theory of elasticity on the various types of structures studied either by the approximate or rigorous methods. One way of proceeding is to measure the functions and reduce them to empirical laws for provisional use, following Newton's plan of "no hypotheses". By the ordinary process of curve fitting, using the theory of elasticity as the limiting case, the true relations are discovered. This is

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 67.

† Washburn's "Principles of Physical Chemistry," Second Edition, see Index.

‡ Perry's "Applied Mechanics," Chapter 15, pp. 366–367. In the author's case the strain is in effect applied at constant temperature, volume, and composition, so that the modulus should be defined at constant pressure and constant density.

§ "The Mathematical Theory of Elasticity," A. E. H. Love, Fourth Edition, Chap. XXI, p. 451, also Chap. XVIII.

similar to the curve-fitting in the theory of columns, in which Euler's theory and the short column formula are limiting cases to fulfill. The method is really what Poincare in his "Theory of Potential" calls a sweeping-out process ("méthode du balayage"); in other words it is one of successive approximations. The laws found, however, are laws in fact of the kind mentioned by Halbert P. Gillette, M. Am. Soc. C. E.*; they agree with the observations and satisfy the equation of motion. Attention should be paid to the fact that the author has developed the right-hand members of the equations of motions or rather the primitive deflections which must be differentiated twice in getting the accelerations, subject to the conditions that his functions must be continuous. When the functions are not absolutely continuous as often occurs in practice they are "smoothed-out" to make them so. As far as the experimental methods are concerned, that of G. E. Beggs, M. Am. Soc. C. E., which is suggestive of the older barrel-hoop method of Eads, leaves little to be desired. Love gives the kinds of equations needed for the arch.† Cylindrical co-ordinates are usually the simplest to use.

In working out correlations with dynamics according to the no-hypothesis method of Newton, it often helps to keep in mind the doctrine of self-similitude enunciated by his contemporary, Swedenborg.‡ As a simple example it may be mentioned that the single potential function, $\frac{1}{r}$ (r = distance from body to point), is fundamental in the theories of astronomy, earth resistance, elasticity, and hydrodynamics, in Osborne Reynold's theory of attraction, and in Dr. Joffé's recent theory of the atom in which direct correlations are carried out consonant with the writer's plan of no-hypotheses.§

Charles W. Comstock,‖ M. Am. Soc. C. E. (by letter).¶—In this paper the author has considered (and to some extent confused) two questions which, in general, are unrelated, namely, the stress-strain relation and the plasticity of materials. The material specifically referred to throughout the paper is concrete but, except for numerical values of certain constants, no reason appears why the arguments should not apply equally to all structural materials.

Rather more than one-half the paper deals with "the proposed theory of elasticity" and illustrative applications. Here, the author has steered wide of Scylla only to be engulfed by Charybdis. He rejects the linear stress-strain relation, but accepts without question the old Bernouilli hypothesis of conservation of plane sections, which not only has no theoretical or experimental justification, but is known to be incorrect except where there is no shear.

This important point is treated very casually. The author attempts no justification of the hypothesis, nor does he write as if he regarded it at all seriously. For example, he states:**

* *Engineering and Contracting*, March, 1929, p. 109, Third column.

† "The Mathematical Theory of Elasticity," A. E. H. Love, Fourth Edition, p. 451.

‡ Emerson's "Representative Men," Lecture III, "Swedenborg."

§ Peirce's "Newtonian Potential Function"; Lamb's "Hydrodynamics," Third Edition, Chapter V; Max Born's "Constitution of Matter," p. 56 *et seq.*; and Joffé's "The Physics of Crystals."

‖ New York, N. Y.

¶ Received by the Secretary, March 27, 1930.

** *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 60.

"From Fig. 6(*b*), it may be seen that, assuming conservation of plane sections during bending,

$$\varepsilon_z = z \frac{d \phi_E}{d s} \quad \text{(12)"}$$

There is no other reference to it in the text. Figs. 6(*b*),* 8(*a*),† 16(*b*),‡ and 17(*b*),§ indicate that the deformation is considered as a simple rotation of plane sections about their respective axes, the positions of which are to be determined. As to Fig. 16(*b*), the assumption is probably correct since there is no shear; as to the others, and, in general, it is not true.

Having selected this starting point the author has thenceforth followed the conventional and usual routine for the analysis of arches, merely substituting for the linear stress-strain relation the exponential relation which was first suggested by Bach as a consequence of his experiments on cast iron.

There is no occasion to comment on the algebraic transformations which lead ultimately to Equations (32).‖ These can contain no more truth than does the assumption on which the entire analysis is founded. It would seem that the author has been misled by the increased complexity of his formulas into expectation of correspondingly greater precision. This is a not infrequent misconception.

The "proposed theory of plasticity" is a repetition of the proposed theory of elasticity with the introduction of a time factor which, however, runs unchanged through the entire analysis to re-appear in Equations (62)¶ just as originally introduced in Equation (34).‡

It is not clear why the "elastic" and "plastic" deformations should have been separated. Assuming the stress-strain relation known it is of no importance whether or not the whole or any part of the strain disappears on removal of the applied forces. The stress distribution is the same in any case.

The expression, "theory of elasticity," originated at a time when somewhat limited experimental data indicated that small strains were elastic in the ordinary sense of the word, and that such strains were directly proportional to the respective stresses.

The mathematical theory of elasticity is a highly developed branch of mathematical physics, singularly free from the speculations and incongruities which permeate so much of modern physical theory. Its fundamental differential equations rest directly on the laws of statics and are no more questionable. In passing from stress to strain it is always carefully stated that linear relationship is assumed. No other assumptions are made. The theory has been extensively and intensively developed through the past hundred years and has been logically set forth in many scholarly treatises in all languages, so that there is now no reason why any interested person should be unfamiliar

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 61.

† *Loc. cit.*, p. 62.

‡ *Loc. cit.*, p. 74.

§ *Loc. cit.*, p. 77.

‖ *Loc. cit.*, p. 66.

¶ *Loc. cit.*, p. 79.

with its possibilities or its limitations. It is equally true whether or not there exists any real material for which the stress-strain relation is linear.

Most of the alleged defects of this theory are chargeable not to inadequacy of the theory itself, but to its attempted application outside its ostensible field or, more often, to the injection of unjustified and unjustifiable hypotheses.

If one has to deal with a material for which the stress-strain relation is known to be other than linear, the difficulties are enormously increased, but no progress can be made by the assumption of stress or strain distributions which do not fulfill the fundamental conditions of the problem.

The equations applicable to a particular problem are sometimes extremely involved. Many years ago the writer convinced himself that, at least in part, this was because of the large number of variables resulting from the use of three components of each magnitude. In view of the directional character of both stress and strain it seemed that quaternion analysis should be an improvement over the usual method of Cartesian co-ordinates. This the writer attempted with some degree of success. The results were not all that were hoped for, but they were encouraging, and it is still believed that Sir William Rowan Hamilton's brilliant conception will afford solutions which have baffled the more common mathematical methods.

It is not clear why the author should have selected 1.3 as his value of m. Mr. Straub states:* "Data obtained from typical experiments by Dr. E. Probst have been used by the writer as a basis for analysis in the basic equation * * *." He then gives equations in which m for compression is 1.105, and for tension, 1.03; yet in all his numerical illustrations he uses $m = 1.3$.

Likewise with respect to p; from the experiments of Mr. Earl B. Smith he finds $p = 1.73$, while from the observations of Freyssinet he concludes, after making an assumption as to shrinkage during setting, that $p = 1.25$. However, he uses 1.3 for numerical illustrations.

As regards q, he finds from the work of Messrs. Fuller and More the value, 0.42, from Smith's experiments, 0.18, and from Freyssinet's observations, after a questionable allowance for shrinkage, 0.40.

Unless the author has a mass of experimental data not cited in his paper, his choice of numerical values would appear to rest on a rather flimsy foundation. If he has such data, full presentation of them, together with the reasoning process by which he determines the most probable values of the constants for use in his formulas, would certainly be of quite as much interest as the formulas themselves.

The form of the author's Equation (34) is open to question. In accordance with it, deformation under any stress whatever must increase progressively for all time, and should end in failure of the structure. It is doubtful whether this conclusion will find general acceptance.

Even if it is true that strain is a function of both stress and time, it is not clear on what ground the author justifies the assumption that it may be expressed as the product of two functions, one of stress only and the other of time alone.

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* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 60.

For concrete, of which the properties certainly change with time, at least during its early history, a more probable assumption would be Equation (33),* in which, m should be a function of the time. The form of the function, m, should be such that it would become practically constant after a finite lapse of time.

It is doubtful if there exist to-day sufficient data of adequate precision to justify an attempt to formulate the stress-strain-time relation for concrete.

The conclusions drawn by the author are based on his formulas. These, in turn, rest on the false hypothesis of conservation of plane sections and on insufficiently established stress-strain-time relations. The conclusions may be true, but they are not proved.

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* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 74.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

LAMINATED ARCH DAMS WITH FORKED ABUTMENTS
Discussion*

By Messrs. Lars R. Jorgensen, William Cain, B. F. Jakobsen, P. Wilhelm Werner, and H. B. Muckleston.

Lars R. Jorgensen,† M. Am. Soc. C. E. (by letter).‡—At and toward the bottom of a high arch dam in a narrow canyon the arch action is inefficient, and the dam has much greater stability as a plug, if the excavation is shaped in such a way that shear parallel to the side-hill can be resisted. The concrete has a very large shearing resistance and it is just as well to take advantage of this.

For fairly thin arch dams the writer would prefer to keep the arch solid in order to preserve full shearing strength in the concrete in all directions. Some difficulties may also arise in getting the laminations to slide on one another, and, moreover, this type of construction increases the form work.

A single trial calculation of the unit shear in the lower portion of a thick arch dam in a narrow canyon will, in general, disclose average shearing stresses along the contact area with rock of less than 100 lb. per sq. in., the assumption being that the plug carries full water pressure. The load is actually divided between arch action and punching shear and since the former is inefficient in this zone, most of the load is carried by punching shear, and it is evident that as long as the unit punching shear is low, the thick arch portion cannot fail. While the shear is normally not uniformly distributed over the contact area, it is believed that some time before failure it will be so distributed.

The ultimate shearing strength of concrete in the lower part of a high arch dam should not be less than 2 000 lb. per sq. in. for good concrete, especially since some arch compression acts more or less perpendicular to the plane of

* This discussion (of the paper by Fred A. Noetzli, M. Am. Soc. C. E., published in February, 1930, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Constant Angle Arch Dam Co., San Francisco, Calif.

‡ Received by the Secretary, February 25, 1930.

shear. A high dam in a narrow canyon should always be investigated for punching shear in its lower portion because that, rather than arch action, is really what supports most of the load. However, it does not make any difference how a dam is held in place as long as it remains there with a sufficient factor of safety.

To overhang the crown of an arch down stream is a practical way to secure minimum length of radii for the lower arch elements. This can be secured to a still higher degree by overhanging the dam on the up-stream side of the abutments as well. Several such dams have been built and are projected.

Such a dam, however, is not a cone dam; both the up-stream and down-stream faces are warped. Merely to incline the dam in the middle does not produce conical surfaces at any place. Fig. 5* shows plainly that the up-stream face projects a maximum at Section *B-B*, and that this projection diminishes toward the abutments where it may become zero or negative. The axis on which the centers lie is not a straight line in any plane and therefore neither face could be a cone as described by the author.†

The author states that‡ "the arches are of uniform thickness at the upper elevations, but are thickened toward the abutments, in the lower portion of the dam." That feature does not make a conical surface out of it. Both faces of this dam are warped in a way that is characteristic of the "constant angle arch" type of dam.

In 1911, the writer gave the type of dam shown in all the illustrations this name. It seemed the best available, in order to emphasize that the important thing was to keep the subtended angle constant and large at all elevations. While the lengths of the radii are variable from crest to foundation, this is not a characteristic but an incident; this fact was the main reason for not giving the name of, say, "variable radius" arch dam.

The Railroad Canyon Dam shown on Fig. 10§ and Fig. 11§, appears to be of the same type and the only thing new on this design is the forked abutments.

It is very possible to construct an arch dam in a V-shaped canyon, such that the up-stream and down-stream faces would be inverted cones. Such a structure could rightly be called a cone arch dam; but it seldom would be practical. Figs. 5 and 7‖ have warped surfaces and are circular, or nearly circular, in horizontal planes, but they are not cone-shaped arches.

WILLIAM CAIN,¶ M. AM. SOC. C. E. (by letter).**—The main object in the proposal of the use of the laminated arch dam, as defined by Mr. Noetzli, is to substitute a series of "thin arches", properly combined, for the "thick arch" as usually constructed. The theory of the thick horizontal arch under uniform normal loads and temperature changes, has been developed by B. F. Jakobsen,

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 272.
† *Loc. cit.*, p. 270.
‡ *Loc. cit.*, p. 288.
§ *Loc. cit.*, p. 289.
‖ *Loc. cit.*, p. 281.
¶ Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.
** Received by the Secretary, March 7, 1930.

M. Am. Soc. C. E.* the writer, in his discussion,† giving an independent analysis of the problem. The application to actual structures often shows that so much tension is exerted at certain points—the extrados at the abutments and the intrados at the crown—that numerous cracks will form there, so that it is proposed to ignore this part of the primary arch that is in too much tension and to substitute for it a so-called "secondary arch". The effect is to place in the dam material that is structurally inoperative, which, Mr. Noetzli claims, is poor design.

The advantage of the laminated dam, is that the corresponding theory is much simpler—approaching that of the "cylinder theory"— but it must be borne in mind that even for the thin arch, cracks are to be expected (as witness the many cracks experienced in the Stevenson Creek Experimental Dam‡), so that the theory that supposes no cracks to form, is again not strictly applicable, even for the thin dam.

In the Stevenson Creek Experimental Dam, cracks occur at parts of the base and at the side walls of the canyon, as well as at other points; so that the dam cannot be regarded as "fixed" over such parts of the base or side walls where cracks occurred. Further, for the upper part of the dam, the arches were deflected down stream for the central portion and up stream for parts nearer the abutments, so that no existing theory applies for much of the dam.

In the theory best known, the dam is considered artificially divided into a series of horizontal arches and vertical cantilevers and the shear between them is neglected, whereas the true solution would be to consider the arched dam as a whole, including the shear, to which the strict theory of elasticity should be applied for a solution. When this is done, doubtless many surprises are to be looked for, because the results will hardly agree with present approximate solutions, especially if the analysis, as usual, proceeds on the hypothesis of no cracks or the dam remaining intact.

The trial-load method has been applied, assuming the usual artificial division of the dam into arches and cantilevers, ignoring the shear between them, and, in addition, regarding as operative only those parts of the sections on which no tension is exerted. This necessarily leads to inexact results, for plain concrete can take a certain amount of tension—possibly 100 lb. per sq. inch, or less. The result would also be inexact if the dam is supposed to remain intact and thus to be able to resist whatever tension would be exerted.

On all these accounts, the problem is seen to be very complex, so that any device that would lead to a simpler approximate solution, should prove welcome. This has been attempted by Mr. Noetzli in his so-called laminated dam, built up by a series of thin dams, to which the theory of thin arches applies, so that the solution, as the thickness of the arch diminishes, approaches that of the cylinder theory. Another great advantage is that the stiff, thick cantilevers of the thick arch, are replaced by thin, flexible cantilevers, which carry very little load, most of it being transferred to the horizontal arches. On that account, possibly the very laborious trial-load method can be omitted and a uniform

* See "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475.

† *Loc. cit.*, p. 522.

‡ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

normal load on the horizontal arches assumed, for which the theory is comparatively simple.

The author further proposes the use of forked abutments, which should give a very stiff structure and thus ensure that the arches are practically "fixed at the ends", and that shorter spans and more favorable radii will result. There are also other advantages, to which the author calls attention, in the proposed sloping of the crown of the arch. It is a question likewise as to whether the arch dam should not be constructed with vertical radial joints at intervals to be subsequently grouted at low temperatures, and also whether the cantilevers should not be simply supported (or hinged) at the base, so as to throw as much of the load as possible on the arches. The writer has given the solution for the latter case as applicable to the Wooling Dam,* using the method† of B. A. Smith, M. Am. Soc. C. E.,

The formulas and diagrams for the computation of the stresses in the thin horizontal arches are based on the writer's theory for the thin arch.‡ For example, the total tangential stress at the point (r, ϕ) is:§

$$P = p\,r - (p\,r - P_0) \cos \phi = p\,r - R_r \cos \phi \qquad (9)$$

Dividing Equation (9) by 144 t, the result is the total rib-shortening stress, in pounds per square inch. In this equation, the first term, $\frac{p\,r}{144\,t} = \frac{p'\,R_u}{144\,t}$, gives the cylinder stress, σ_c, of the author's Equation (2)‖; the second term, $-\frac{H_r \cos \phi}{144\,t}$, gives the stress due to the pull, H_r, at the crown, tending to lengthen the neutral axis. The algebraic sum gives the total rib-shortening stress.

Assume, for convenience, that $y = \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi\right)$; then the moment at (r, ϕ) is:§

$$M = R_r\,r\,y \qquad (10)$$

so that the stress due to this moment at the intrados or the extrados, in pounds per square inch, is:

$$\frac{1}{144}\,\frac{6\,M}{t^2} = \frac{1}{24}\,\frac{H_r\,r\,y}{t^2} \qquad (11)$$

Hence, the stress at intrados or extrados, due to the pull, H_r, and the moment is,

$$\sigma_r = -\frac{H_r \cos \phi}{144\,t} \pm \frac{H_r\,y\,r}{24\,t^2}$$

This is the author's Equation (5),¶ in which $\phi = 0$ at the crown and $\phi = \phi_1$ at the abutment. The total unit stress, is thus, $\sigma = \sigma_r + \sigma_c$, as in the author's Equation (8).**

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 311.
† *Loc. cit.*, Vol. LXXXIII (1919-20), p. 2027.
‡ "The Circular Arch Under Normal Loads", *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.
§ *Loc. cit.*, p. 236, Equation (4).
‖ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 265.
¶ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 237, Equation (13).
** *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 266.

Similarly, the results for temperature changes are derived for the free arch, not including the effect of the restraint due to cantilever action. The formula given for σ_r is not the rib-shortening stress, which is strictly,

$$\sigma = \frac{H_r \cos \phi}{144\, t} \qquad (12)$$

and is uniform over the section of thickness, t.

The author gives in Fig. 9,* a plan of a French dam consisting of five, thin, reinforced arches, placed one below the other, 60 ft. or more apart, the heights decreasing down stream. The principle of lamination is well illustrated in the design which has many advantages and some disadvantages. As to the latter, the extra form work is objectionable, and possibly the arches are not fixed at the abutments so well as in the author's design. His laminated arch is very interesting, and it is to be hoped that it will be fully tested in an actual construction.

B. F. JAKOBSEN,† M. AM. SOC. C. E (by letter).‡—The "laminated cone arch dam" appears to be a constant-angle arch dam,§ provided with one or more asphalt-covered slip-joints, which are concentric with the up-stream face of the dam. These joints are assumed to permit minute movements and adjustments between the separate arch sections.‖ The author is so certain that this must happen, that he suggests a test of the factor of safety be made by loading each arch lamina of a model separately.¶ That process amounts to taking for granted just what is most in need of proof, namely, whether or not these slip-joints, the areas of which must be measured in acres, will function as assumed. A test on a model would give no reliable information as to whether or not several acres of asphalt-painted surfaces will slide more or less freely on each other; furthermore, it would give no information as to how aging, temperature variations, etc., would affect the coefficient of friction. The proposed design has no application to small structures, since there is no decided advantage in laminating the arch unless it is quite thick. For comparison, it may be stated that the up-stream face of the proposed San Gabriel Dam of the Los Angeles County Flood Control District, has a surface area of more than 20 acres.

The author refers** to the successful testing of the Stevenson Creek Arch Dam and suggests that a simple test on a small model would furnish the factor of safety of his proposed design. A comparison of the design data for the Stevenson Creek Dam with the test data, and the values obtained from the celluloid model, suggests caution rather than confidence.†† There appear to be too many factors which the model may not reproduce correctly, but which would affect the safety of the structure.

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* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 285.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, March 25, 1930.

§ "The Constant-Angle Arch Dam," by L. R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 685.

‖ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 277.

¶ *Loc. cit.*, p. 280.

** *Loc. cit.*, p. 262.

†† *Loc. cit.*, May, 1929, Papers and Discussions, p. 1241.

Mr. Noetzli assumes a coefficient of friction of 0.5 for the asphalt-painted joints.* This may be a fair assumption under ideal conditions, for small areas, while the asphalt is fresh and its temperature not too low; but such ideal conditions, involving practically true surfaces, as in a machined joint, will not be obtainable in large dams. Furthermore, the effect of shrinkage, swelling, temperature variations, etc., will warp these surfaces, and the total result is likely to be high compression between the two surfaces at various points and no compression at other points. This will tend to increase the frictional resistance.

The friction here involved is that of rest following a considerable period of no movement, during which time the surfaces have been pressed tightly together, and the movements are minute, as stated by the author. These are the conditions most unfavorable for a small coefficient of friction.

The writer is not able to follow the author's calculations of the influence of friction,* but if any friction whatever exists, it will defeat the purpose of the lamination to some extent. It is also clear that there will be no movement at all, where the shear is less than the product of compression between the surfaces and the coefficient of friction. That the average shear for the whole surface is greater than the average frictional resistance, will hardly suffice, because relatively small movements at places where there is an excess of shear will materially decrease that shear. Such movements will not greatly affect the frictional resistance, because the compression between the surfaces will not be materially altered. At the crown section the shear is zero and the compression is considerable; there is no need for motion between the surfaces right at the crown section; but immediately outside, a relative movement is required and the shear is still very small.

As an actual example the writer has considered an arch with the ratio, $\frac{t}{r} = 0.5$, and the central angle, $2\,\phi_1 = 120$ degrees. Fig. 12(*a*) shows the radial compression, σ_r, at the crown section, where the shear is zero, Fig. 12(*b*) shows the radial compression and the shear stress, τ, for the section determined by $\phi = 30°$, and Fig. 12(*c*) shows the same for $\psi = 60$ degrees. These values were calculated from the writer's paper, "Stresses in Thick Arches of Dams",† on the assumptions that Poisson's ratio may be neglected that is, that $m = \infty$; that the arch carries a load of 200 ft. of water, or 87 lb. per sq. in.; and that shrinkage, swelling, temperature variations, and the yielding of the abutments are negligible. The effects of Poisson's ratio, yielding of the abutments, and swelling, are probably to increase the radial compression and to decrease the shear, while the effect of shrinkage is the reverse.

An arch 50 ft. thick would presumably be divided into three laminæ.‡ Fig. 13 shows the values of σ_r, the radial compression and of τ, the tangential shear, at points 16.5 ft. from the up-stream face of this arch, and Fig. 14 shows

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 280.

† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 475.

‡ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 283.

the ratio of $\frac{\tau}{\sigma_r}$. Let f equal the coefficient of friction; then no motion will result if $f \geqq \frac{\tau}{\sigma_r}$.

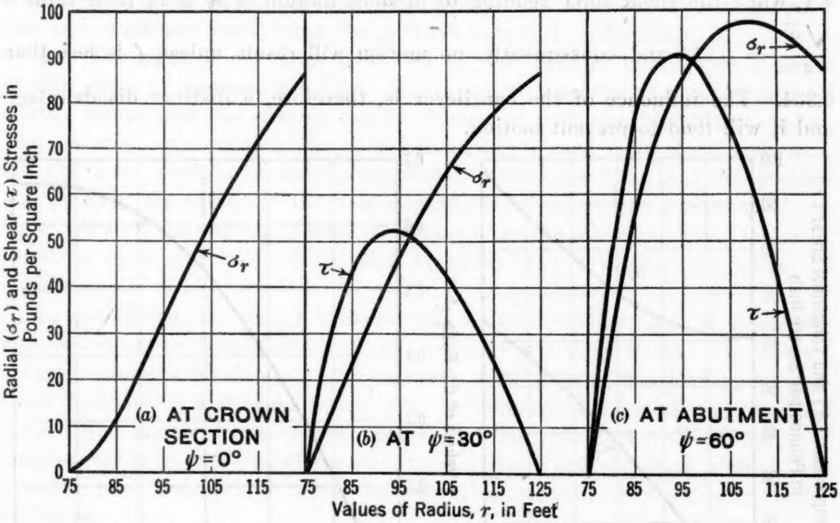

FIG. 12.—RADIAL AND SHEAR STRESSES IN AN ARCH 50 FEET THICK.

For values of $f \geqq \frac{\tau}{\sigma_r}$, motion will result (for the moment, disregarding the stresses in the cantilever), and the stresses will be partly re-adjusted. The re-adjustment will be complete only if $f = 0$. Fig. 14 shows that, for $f = 0.5$ (as assumed by the author*), $f \geqq \frac{\tau}{\sigma_r}$ from the crown section where $\phi = 0$, to $\phi = 28$ degrees. For angles greater than $\phi = 28°$, motion and a slight re-adjustment of stresses should result. This re-adjustment will be adversely affected by the half of the arch where the shear is insufficient to produce any movement. The average value of $\frac{\tau}{\sigma_r}$ for the whole arch is about 0.483, which is less than f. For these reasons the writer can not accept the author's conclusion, that the friction in the joint is negligible.* On the contrary, it seems, if not evident, at least highly probable, that the friction is sufficient to prevent all but very minor adjustments of the stresses, and it would also appear that this is the safe and conservative assumption.

Mr. Noetzli appears to have overlooked the effect of the cantilever. The radial compression at any point in the dam is the algebraic sum of the compression due to the arch and that due to the cantilever, while the tendency to produce motion, is the geometric sum of the arch and cantilever shears. As an illustration, assume arbitrarily that at any point of the arch the shear is τ

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 280.

and that the arch radial compression is $2\,\tau$. Then, if $f < 0.5$, motion will result. If, at this same point, the cantilever shear is also τ and the cantilever radial compression is $2\,\tau$, the incipient motion is in a direction making an angle of 45° with the horizontal; but the resulting radial compression is now $4\,\tau$, while the shear force tending to produce motion is $\sqrt{2}\,\tau$; their ratio is $\frac{\sqrt{2}}{4} = 0.354$ and, consequently, no motion will result, unless f is less than 0.354. The influence of the cantilever is, therefore, a distinct disadvantage and it will tend to prevent motion.

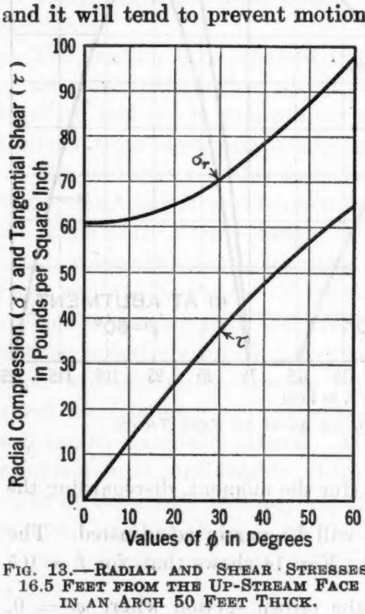

FIG. 13.—RADIAL AND SHEAR STRESSES, 16.5 FEET FROM THE UP-STREAM FACE IN AN ARCH 50 FEET THICK.

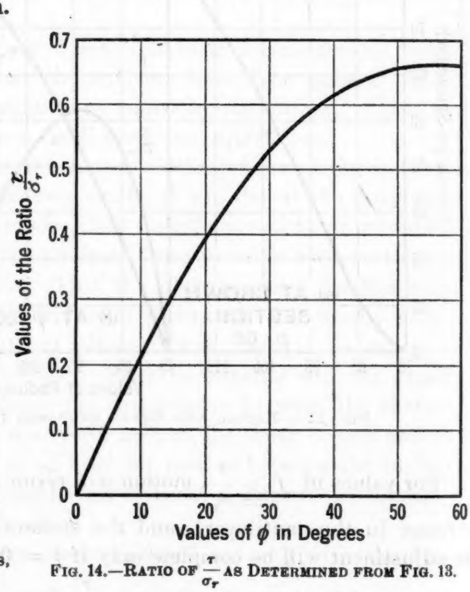

FIG. 14.—RATIO OF $\frac{\tau}{\sigma_r}$ AS DETERMINED FROM FIG. 13.

The object of the theory of the secondary arch is not to determine the actual stress distribution,* but to establish a limiting maximum value of the stresses, as the writer has pointed out.† The author is quite correct in stating that a thick arch involves inherent complexities and uncertainties;‡ but these, the writer believes, are insignificant compared to the complexities and uncertainties involved in determining the degree of stress adjustment brought about by the laminations, because this involves everything that concerns a thick arch and a thick cantilever, with the added complications of small local movements and local stresses at the joints and the uncertainty of the coefficient of friction. These laminations, like the devices for underdraining a gravity dam, may work to some extent, and they may not; the safe assumption is that they may not function.

The various suggestions of joints and hinges, which have been proposed during the last decade, do not appeal to the writer. Shrinkage (which is pos-

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 262.
† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 510 and 576.
‡ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 263.

sibly the worst feature of concrete arch dams) can best be counteracted by some method of grouting the contraction joints, such, for example, as was proposed by L. R. Jorgensen, M. Am. Soc. C. E., in a paper entitled, "Improving Arch Action in Dams".* This method has been used in connection with a number of dams and apparently with considerable success. It is more positive and much less costly than the author's proposed laminations.

The Dordogne River Dam described by the author† consists of a series of thin dams of decreasing height. There are no uncertainties occasioned by sliding joints, and the stresses can be calculated with considerable precision. The material required is no doubt less than that needed for a thick arch dam, but the form work is materially increased. The objection, that the loading and possibly the safety of the dam depends upon the proper handling of the water between the various arches, is obvious. On the other hand, the factor of safety of each arch is no doubt high (possibly six) if there is no danger of buckling.

In conclusion, the writer believes that an arch thicker than is absolutely required is not an unmitigated evil nor necessarily a mere waste. The outside concrete protects the interior load-carrying portion from weathering, temperature fluctuations, and rapid drying out. In addition, if a thick arch is stressed beyond the elastic limit of the concrete at any place, a redistribution of stresses will take place, as discussed by the writer,‡ and the thick arch will be found to have a materially higher load-carrying capacity than ordinary theory indicates.

P. Wilhelm Werner,§ Assoc. M. Am. Soc. C. E. (by letter).‖—The author aims at a rational use of the arch principle through application of arch lamination and of forked abutments. The views expressed in the paper are of much interest to the writer because of certain investigations made along the same lines.

Concerning the arch lamination, the writer believes that the design, from a theoretical point of view, is a step toward the perfect arch dam. As laid out by Mr. Noetzli, the construction is also practicable and economically feasible.

As regards the forked abutments, the main question seems to be to provide for monolithic action of the abutments. In general, it is probably possible to insure safety of the abutment in regard to sliding, overturning, and other rules governing the design of a gravity section, provided the abutment could be assumed to act as a monolithic structure. In the writer's opinion it is questionable, however, whether it is possible to accomplish this with either the forked abutment or the so-called gravity tangent.

Referring to Fig. 5,¶ the abutment of Dam No. 1 has a length of about 250 ft. and a height of about 75 ft. It is generally conceded** that a straight

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

† *Loc. cit.*, p. 284, and Fig. 9.

‡ *Loc. cit.*, Vol. 90 (June, 1927), p. 507, Article XVII, "Stress Distribution at Failure".

§ A. B. Vattenbyggnadsbyran, Stockholm, Sweden.

‖ Received by the Secretary, April 8, 1930.

¶ *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 272.

** *Loc. cit.*, p. 372.

gravity concrete structure with these dimensions would inevitably show transverse cracks in the masonry, unless contraction joints are provided at suitable intervals. However, if cracks should occur, or if the transverse joints should open up, it is easily conceivable that the abutment would not be fit to transmit the thrust to the side-hills. Even if pressure groutings are applied, it does not seem to be an ideal way of finishing an arch dam. Unless special precautions are taken to insure monolithic action of the abutments, or unless the conditions at the site are specially favorable for their construction, the writer does not feel inclined to rely upon either the forked abutment or the gravity tangent for transmitting the arch thrust to the rock. This point should receive careful consideration. It would be interesting to know how the problem has been solved in the cases referred to by the author.

H. B. MUCKLESTON,* M. AM. SOC. C. E. (by letter).†—Two planks side by side will carry twice as much load as one plank. If one is laid over the other, with rollers between, the combination will still carry twice as much. If the two are in contact, the combination will carry rather more than twice and, if the two are clamped together, the carrying capacity will increase until, when the friction between them exceeds the longitudinal shear in a simple beam of the same depth, the combination will act as a simple beam and carry four times the load.

The author's expedient of laminating an arch dam in order to avoid the disadvantages of the thick arch will be successful according as he can avoid the frictional resistance between the laminations. This he proposes to do either by lubricating the surfaces by creating a film of asphalt, or by a physical separation of the surfaces, using hydrostatic pressure to transfer the proportionate part of the load from one lamination to the next.

The first method would be excellent provided the surfaces were truly circular in horizontal section with no departure from the true circle exceeding one-quarter, or thereabouts, of the thickness of the lubricating film. It is more than doubtful whether a large dam could be built with no departure greater than ½ in., which means that the asphalt film would be at least 2 in. thick if the benefit is to be realized. This is many times the thickness of the capillary film, even for asphalt, and, as a consequence, the film would exert hydrostatic pressure equal to its full height (even when it appears to be brittle, asphalt is a highly viscous liquid). This would act to transfer the proportional part of the load when the reservoir was full; but when the reservoir was empty, it would press the up-stream lamina in a radially upstream direction with a greater tangential tension as a result than the material could stand, unless it were heavily reinforced for the special purpose.

The second method, by a physical separation using water as a medium to transfer the load, would work when the reservoir was full; but when the reservoir was empty, the space between the laminæ must be emptied also, and this means manual or automatic control. In either case, such factors as silt, débris, ice, and deliberate human intervention—which is by no means im-

* Cons. Engr., Vancouver, B. C., Canada.

† Received by the Secretary, April 8, 1930.

possible—offer too many possibilities of trouble to make the idea confortable. The thought of deliberate human intervention may seem like borrowing trouble, but it must be remembered that the "cranks" and anarchists are not all dead by any means. It is not long since that one of them tried to wreck the Welland Canal for apparently no good reason, while, during the World War, enemy sympathizers on more than one occasion undertook the deliberate destruction of bridges and other works in Canada.

In one respect, the paper is to be commended. Mr. Noetzli seems to have avoided successfully the idea of wasting acres of paper on the vain attempt to divide the load between the arch and cantilever elements to nine places of decimals. This extreme exactness has always appealed to the writer as an attempt to split a mathematical hair, with the purpose of adding the weight of the left half to that of some saw-log the weight of which has to be guessed anyway.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE CHESAPEAKE AND DELAWARE CANAL
Discussion*

By F. T. Chambers, M. Am. Soc. C. E.

F. T. Chambers,† M. Am. Soc. C. E. (by letter).‡—In recording the construction stages of the Chesapeake and Delaware Canal and in giving quantities, yardage, costs, and methods in connection with the dredging operations of the conversion from lock-level to sea-level, the author has done a useful service. The description of the deposit of spoil in the high-level disposal areas by means of modern hydraulic dredges of standard construction and the record of costs of this part of the work are in themselves sufficient justification of the paper without its many other features. The dredge performance is the more creditable by reason of the condition of the material as encountered.

While the history given is sufficient as to the physical aspects and the Canal Company finances, a bibliography of all publications concerning this canal and the many projects in general for canals across the Delaware Peninsula should be attached to this paper. Several public documents treat of this subject. A single report§ relates much of this effort and records other reports with borings along this and other routes.

Public Joint Resolution No. 37, approved June 28, 1906, authorized the President of the United States to appoint a commission consisting of an officer of the Engineer Corps of the Army, an officer of the Navy, and one person from civil life,

"* * * to examine and appraise the value of the works and franchises of the Chesapeake and Delaware Canal, connecting the waters of the Chesapeake and Delaware bays, with reference to the desirability of purchasing

* This discussion (of the paper by Earl I. Brown, M. Am. Soc. C. E., published in February, 1930, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.

† Rear-Admiral (C. E. C.), U. S. N.; Public Works Officer, Third Naval Dist., New York, N. Y.

‡ Received by the Secretary, March 11, 1930.

§ Senate Doc. No. 215, 59th Cong., 2d Sess.

the said canal by the United States and the construction over the route of the said canal of a free and open waterway having a depth and capacity sufficient to accommodate the largest vessel afloat at mean low water;" and, also, "to the extent that the same can be done from the surveys heretofore made under the direction of the War Department and within the limits of the appropriation herein made" to "examine and investigate the feasibility, for the purpose of such a waterway, of the route known as the Sassafras route."

The Commission was ordered to:

"Make a report of its work, together with its conclusions upon the probable cost and the commercial advantages and the military and naval uses of each of the said routes, to the Secretary of War, who shall transmit the same to Congress at its next session."

The late Gen. Felix Agnus, of Baltimore, Md., was the Chairman and Maj. (late Maj. Gen.) C. A. F. Flagler, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and the writer were the other members of that Commission. From the very nature of its precept, the Agnus Commission was obliged to use past reports and other available data, and thus Senate Document No. 215 contains, in addition to survey and boring figures resulting from the field work of its own personnel, much that appears in previous reports.

Although it was the apparent belief of many interested people that the Chesapeake and Delaware Canal should be taken over by the National Government and converted to sea-level operation by stages as the traffic demand might require, it will be seen from the text of the Congressional Resolution that the Agnus Commission started with a considerable handicap to full investigation. It was limited not only in time and money, but more particularly in the requirement of a canal to accommodate the largest vessel afloat. Although the Commission took considerable liberty in its interpretation of that expression it was still obliged to consider a depth of 35 ft. at mean low water. True, it took further liberties and figured a 30-ft. depth also but, because of its instructions, it could not estimate on barge depths. It did, however, go so far as to recommend the purchase of the Chesapeake and Delaware Canal for a sum not to exceed $2 514 289.70 and to record the public pressure for gradual deepening. It may be stated here that the Commission assumed that much of the high elevation excavation would be steam shovel work.

Although the Commission reported upon the Sassafras route, it found in favor of the Chesapeake and Delaware route, recommending a sea-level canal following much the same lines as that described by the author, but debouching upon the Delaware several thousand feet below Reedy Point. It also found for jetties at the Delaware entrance "for at least 1 000 ft. from shore".

The author's study* of the canal hydraulics is interesting. It is to be hoped that observations of actual tidal heights and currents will be made and published. The Angus Commission gave consideration to the matter of currents and to a difference of tidal heights between the Chesapeake and the Delaware ends of 6 to 10 ft. It also gave consideration to the tidal lag at the Chesapeake end. It compared the physical conditions at the Chesapeake and Delaware Canal with those at Suez, where M. Quellenec reported an

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 335.

extreme tidal difference of 9.02 ft. and a resultant current of 2.6 knots. Furthermore, it cited the lockless sea-level canals of Corinth and Tasmania, which are easily traversed by vessels, together with computations made for a tide-level Panama Canal. It did not attempt, on account of its limited time and funds, to make complex computations based on any recognized formula, but did make rough over-all estimates. It concluded that a tide lock was not indicated as part of the cost.

The writer has nothing to criticize in this able paper. He agrees with the author in the conclusion* that in future enlargement of the canal it may be a saving of expense in the lessening of landslides to remove some part of the overburden by dry methods.

His only disagreement is with regard to wet borings.† Such borings are not absolutely unreliable for determining the character of the subsoil. The writer has made or supervised the making, first to last, of many borings at numerous places. He does not choose wash-rod borings where time and money will procure borings of a more positive type. A small percentage of his borings have been exclusively wash borings. Many wash borings are worthless, or nearly so; but the wash rod with casing can be applied with considerable satisfaction for the determination of some subsoil conditions, especially of conditions for dredging and where due care is had in the use of sampling devices, ancillary to the methods of the wash rod.

* *Proceedings*, Am. Soc. C. E., February, 1930, Papers and Discussions, p. 339.

† *Loc. cit.*, p. 340.

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PAPERS AND DISCUSSIONS

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RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION: A SYMPOSIUM

Discussion*

By Messrs. R. B. Kittredge, Henry A. Palmer, and J. Parker Snow.

R. B. Kittredge,† M. Am. Soc. C. E. (by letter).‡—At this time, when extravagant claims both for and against inland waterways are being so freely made from numberless platforms, and so widely discussed in the public press, there rise in many minds questions of reasonable doubt as to whether the further development of inland waterways would result in a waste of the taxpayers' money or in a substantial means of relief to a distressed Middle West and to a depressed agricultural industry. The paper by Mr. Cornish§ presents the opportunity for thoughtful discussion by the profession, which should do much to clarify the situation.

One of the conclusions of this paper is that, after the completion of waterways now authorized, the Mississippi Valley Inland Waterway System by 1950 could produce annually the equivalent of 40 000 000 000 ton-miles of railway transportation at a minimum economic advantage of $160 000 000 per annum. If true, this is important.

Apparently, however, the conclusion has been drawn as the result of an analysis which includes two errors of major importance: First, figures which cover the total cost of railway transportation have been used in comparison with other figures which include only a part of the cost of waterway transportation; and, second, the railways have in effect been charged not only once but twice with the interest on a substantial part of their investment. When these interest charges are estimated correctly, and when the costs compared

* Discussion on the Symposium on Relation Between Rail and Waterway Transportation, continued from April, 1930, *Proceedings*.

† Prof. of Transportation Eng., State Univ. of Iowa, Iowa City, Iowa.

‡ Received by the Secretary, March 8, 1930.

§ *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 561.

are the total costs, the logical conclusions as to the economic advantage of completing this waterway system may of necessity be materially modified.

Mr. Cornish has estimated* at $1 880 000 000 the investment required to expand railway facilities for an additional 40 000 000 000 ton-miles of railway traffic. He has also estimated at $600 000 000 the investment necessary to supply an equivalent amount of waterway transportation by the completion of certain partly constructed waterways in the Mississippi Valley. Capital to the amount of $1 280 000 000 is thus assumed to be saved for investment elsewhere if, by the completion of these waterways, the indicated expansion in railway facilities is indeed avoided. At this point, by adding to an estimated $90 000 000 annual saving in freight charges the sum of $73 600 000 to cover the annual interest, at 5¾%, on this additional investment of $1 280 000 000, Mr. Cornish arrives at a figure of $160 000 000—this representing, in round numbers, the minimum yearly economic advantage of waterway transportation on this Mississippi Valley System. In this computation consideration has nowhere been given to a considerable part of the cost of waterway transportation, and interest charges have been handled incorrectly.

The author's $90 000 000 of annual saving in freight charges was estimated by using rail rates of 11 mills per ton-mile, and water rates of 80% of the equivalent rail rates. The rail rates thus used would cover the total cost of the rail transportation, including taxes, all railway maintenance costs, and interest on the total railway investment. On this waterway system, rates fixed at 80% of the equivalent rail rates could not be expected to cover the total cost of the waterway transportation, including an allowance for taxes, the maintenance costs of the waterways, and interest on the additional investment required to complete their construction.

For some time past, the average railway freight rate has been approximately 11 mills per ton-mile, and railway revenues over this period have covered the total cost of railway transportation, including taxes, and all maintenance costs, and whatever interest has been paid on railway investment. On inland rivers and canals, with the exception of a part of the Ohio River System and of the connecting channels of the Great Lakes, water rates at 80% of the equivalent rail rates have not been sufficient to cover the total cost of the waterway transportation. The basis for such rates is what the traffic will bear and is not what the transportation by water actually costs. Low rates on inland rivers and canals have been possible only because the taxpayers have constructed and maintained the waterways and have also waived taxes and interest upon them. If the railways were similarly relieved of maintenance, taxes, and interest, railway rates might well be cut in two. This does not mean that transportation development on inland rivers is never justifiable, but it does mean that, before its economic justification can be established, full consideration must be given to its total cost.

When waterways are constructed and maintained at Governmental expense, the saving to the shippers involves a sacrifice by the taxpayers which by no means should be overlooked. Maintenance costs of the waterways, interest on their construction cost, and taxes not assessed, are all matters of more than

* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 580.

academic interest and, because this is true, they all deserve careful consideration. Although a part of the cost of waterway transportation may be concealed for a time, in the end it cannot be escaped. The burdens on the taxpayers may be less in evidence than are the savings to the shippers, but, by virtue of their being less apparent, they are in no sense less real.

Instead of deducting from the $90 000 000 estimated annual saving in freight charges the sacrifices required of the taxpayers to make this saving possible, Mr. Cornish has added $73 600 000 as representing 5¾% interest on $1 280 000 000 of the railway investment.* This interest has been added in spite of the fact that, when railway rates were estimated at 11 mills per ton-mile, interest on all the railway investment, including this $1 280 000 000, had already been charged once against the railways. Mr. Cornish clearly estimated that railway rates of 11 mills per ton-mile would be sufficient to cover interest on all the railway investment. Interest on a part of this investment should not be charged against the railways again.

If, by constructing waterways, $1 280 000 000 of capital, which would otherwise be invested in railway facilities and earning 5¾% interest, could indeed be released for investment elsewhere, some benefit would undoubtedly result to the credit of the waterways, although such a benefit would not amount to any substantial part of $73 600 000 annually. The benefit of releasing for investment elsewhere capital already earning 5¾% interest would not amount annually to another 5¾% of the capital.

Wherever the saving in freight charges is less than the sacrifice required of the taxpayers, the construction and operation of inland waterways would result in an increase in the total cost of transportation. In the face of this condition, the economic justification of the waterways must be based squarely on some benefit in addition to the saving effected in freight charges.

Other benefits, in addition to the savings in freight charges, and at the same time certain other disadvantages, in addition to the contributions by the taxpayers, would follow the development of an inland waterway system. All these should, of course, be carefully evaluated before drawing any final conclusion as to the economic justification of the waterway transportation.

Mr. Cornish has clearly formulated certain definite questions with reference to railway and waterway transportation, and he has courageously given both his own answers to these questions and the process of reasoning which lay behind them. Inevitably some errors will creep into so comprehensive a discussion; and the writer believes that errors have been made which result in overestimating the value of waterway transportation. Errors which may exist in the paper, however, should not be allowed to obscure all of value which there remains. It is to be hoped that the paper by Mr. Cornish will receive the full and intelligent discussion which it so richly deserves and that as a result will come a much clearer comprehension of the relative economic advantages of transportation by water and by rail.

The completion of this Mississippi Valley Inland Waterway System may be justifiable; the advantage, however, from a purely economic standpoint, of its immediate completion and use has not to the present time been convincingly demonstrated.

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* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 580.

HENRY A. PALMER,* ESQ. (by letter).†—An understanding of a few fundamentals should precede any consideration of inland waterway development, or the lines along which such development should proceed. Some of them do not seem to be understood generally.

The movement for the development of inland waterways as an agency for the transportation of freight is due to (1) the passion of the engineer for the utilization of natural waterways; (2) the desire of certain localities and business interests for lower freight rates—no matter at whose expense; and (3) the willingness of demagogic politicians always to be "for" what they think their constituents desire.

Any policy of this sort, however, should be based on something more sound. There is nothing sacred about any kind of transportation and, if it is found that water transportation fills a need that can be supplied in accordance with sound and reasonable economy, it should, of course, be supplied. Indeed, it will be supplied, because capital is constantly on the lookout for such opportunities; but it ought not to be supplied under any form of paternalism or conditions that require support to be furnished by the public in order that those who benefit may prosper.

Thus, wherever there is, on adequate and fair investigation, found to be a reasonable demand for the development of a waterway and a reasonable assurance that traffic will seek it, once it is developed, the Government should undertake to develop that waterway. The development, however, ought to be with the understanding that operators of transportation on it shall be subjected to the same sort of regulation that governs their competitor, the railroad; that rates shall be based on a real cost study, which has never been made; and that operators shall pay into the public treasury a reasonable fee for permission to use the waterway as a place of doing business for profit.

Under present conditions inland waterway operators are permitted to make rates arbitrarily 20% less than rail rates. There is no economic basis for that differential; that is, no one knows just how much less the cost of water transportation is than that of rail transportation, if any. Waterway carriers make their own regulations and do things for shippers that the railroad companies, under strict regulation, are not permitted to do.

Furthermore, whatever else may be permitted, the Government should retire from the transportation business of the Mississippi and Warrior Rivers. Government in business of any sort is not in accord with approved public policy. In this particular case it causes sad complications because the Government corporation that operates this business insists on showing a profit, and that profit is computed with no overhead expense, such as a private operator must set up—no taxes, no insurance, and no cost of capital. For that reason, it is not a fair exhibit of what can be done in waterway transportation, although Government operation on these waterways purports to be an experiment for the purpose of demonstrating to private capital what can be done. Of course, private capital will not be fooled, but the public is fooled and has an extremely

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† Received by the Secretary, March 21, 1930.

false idea of the cheapness of water, as compared with rail, transportation. This confused idea prevails even in Congress.

Much is said in favor of developing waterway transportation on the theory that the railroad companies may not be able to take care of the country's growing traffic. There can be little to this if the Government carries out the spirit and even the letter of the Transportation Act of 1920, which is that the railroads shall be enabled to earn a fair return on the money invested in them. If they are permitted to do this they can be counted on, in their own interest, to make the extensions necessary to care for increasing business.

Admit for the sake of the argument, however, that the railroads may not be able to cope with increasing traffic demands, as time goes on, either because they are incompetent or because they are so restricted in earnings as not to be able to do the things that ought to be done and that they would like to do. Inland waterway development might then become advisable; but why develop the waterways under unfair conditions, even then? Why not place them on a sound economic basis at once, instead of providing here and there facilities for those who can use them at the expense, in part, of the people as a whole? Why not make those who use them pay for what they get? Certainly, no one in his senses has ever advocated such a system with respect to rail transportation; but, when the subject is carefully thought out, if the Government concludes that it must provide cheaper and more transportation facilities, why should it not build a railroad, for instance, between Chicago, Ill., and New Orleans, La., charge rates 20% less than the rates it permits privately owned railroads to charge, pay the deficit out of the public treasury, and then tell the people not only that it is giving them low rates, but that the operation is profitable? Any one capable of thinking can see that such a thing would be unfair to the railroad companies, subversive of sound transportation policy, and indefensible in economics. Yet what is the difference, in principle, between such a course and the one that is being pursued?

Selfish and unsound views with respect to this question may be expected from those who have interests at stake—the railroad companies, on the one hand, who do not want this competition, and shippers, on the other hand, who can use waterways and can profit by the low rates offered. Engineers, economists, and statesmen, on the other hand, ought to clarify their views and act in accordance with common honesty and sound practice.

J. Parker Snow,* M. Am. Soc. C. E. (by letter).†—The Symposium sets forth an imposing array of data on various types of transportation as well as many valuable comments on the ethics of Government improvement of public ways by land and water. The discussion here concerns the paths or ways of transportation rather than the substance carried or the commercial aspects of the great subject; and railroads and waterways are the special ways in question. Passenger traffic on these ways seems to be a decreasing factor at present, and freight traffic only will be discussed herein.

To treat the subject rationally it must be recognized that there are two fundamentally different classes into which these ways naturally divide. Rail-

* Cons. Engr., Boston, Mass.

† Received by the Secretary, April 5, 1930.

roads are economically essential monopolies, privately owned and operated, while waterways are of two kinds. Artificial canals, whether owned privately or by the Government, are properly, like railroads, essential monopolies; while the sea, lakes, and navigable streams are, like highways, properly free for all who furnish the vehicles which they use. Ways of the first kind are common carriers and should be under public regulation. They are commercial ventures; no one but their owners can use them, and the traffic they carry must pay rates for the service. On the other hand highways and natural waterways are open to free competition to all proper use. They are not, speaking broadly, well adapted for use by common carriers. General Ashburn intimates this as applying to waterways. His Table 1* shows a slight net income in common carrier service on waterways for 1928; but, if a fair return on cost of equipment and for depreciation had been included, the net would have been in red type. If freight rates were computed on a rational basis, a just comparison of them could be made which is impossible under the present American method. The German method for a generation or more has been to fix the rate on the sum of the costs at each end of the run and that of the line haul. If this method were in use in the United States much fairer comparison could be made than with the rates now existing. Street-car lines occupy highways and are common carriers, but they furnish their own track and vehicles and are necessarily in the first classification. Even buses must be so classed if used as constituent parts of car lines or railroads, but sound judgment is needed in granting or denying permits for competing bus lines.

Ways of the second class must be adapted and maintained by the public. Municipalities, with assistance from the State and Nation, handle highways, while the Federal Government maintains the waterways. Proper maintenance means improvement to keep step with advancing means of transport. The sums spent to make and keep harbors and navigable rivers properly usable seem enormous, but they are small compared with the vast aggregate sum spent on highways. The cost of improving highways can happily be kept reasonably low to the common taxpayer by means of the gasoline excise. Improvement of natural waterways must be done by the Government; it is not subsidy; it is justifiable improvement, and there seems to be no way to pay for it except by appropriations from the Treasury. Unhappily, the securing of such appropriations sometimes leads to "log-rolling bees" in Congress. Whatever the difficulties may be, however, public ways must be kept abreast with the march of progress.

Railroads are old; vast progress has been made in developing means of transport since their inception. Naturally, these new means encroach on the business that the railroads would like to handle, but, if the railroads cannot do the work better or cheaper than the new means, they will be passed by just like the canal-boat, the freight wagon, and the stage coach a century ago. The railroads, however, are still the backbone of this country's interchange of goods. They should be free to use public ways by land or water if they can compete with the open competition naturally inherent to such ways.

**Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 559.

Motor vehicles adapted for use on highways have changed the present business status. This may be called an oil-traction age. If railroad heads would give more effort to develop oil-electric locomotives to replace steam traction instead of expending their energies in the contest to be the "big boss" in the preparation for consolidation by straw corporations and other means, the case of the railroads might look better. For instance: If a train-load of goods stands at St. Louis, consigned to New Orleans for export or for domestic distribution; and if waterway transport by means of the magnificent channel maintained for the public by the Government is enough cheaper than rail transport to pay a railroad for installing appliances at both ends of the route to transfer cars between land rails and rails on car-floats, said floats and their propelling power being the property of the said railroad, then, the railroad should be allowed to establish such facilities, not as a monopoly, but on the same basis as railroad trucks are now allowed to distribute goods *via* city streets. If this type of transport is cheaper than the established rate by rail, it should be the base for the standard rate to be applied to goods of the class that can admit the slower rate of movement. A rate of 80% of the rail rates is unscientific and absurd.

Motor vehicles adapted for use on highways have [illegible] the [illegible] [illegible]. This may be called [illegible]. If railroad [illegible] would give more effort to [illegible] [illegible] instead of extending their energies [illegible] to be the "big [illegible]" [illegible] the registration for consolidation [illegible] and other [illegible] [illegible] of the railroads might [illegible]. If a [illegible] [illegible] at St. Louis, [illegible] [illegible] [illegible] [illegible] [illegible] and maintained for the public [illegible] and wheels.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

PREPARED BY COMMITTEES FROM THE AMERICAN SOCIETY OF CIVIL ENGINEERS AND THE AMERICAN RAILWAY ENGINEERING ASSOCIATION

Discussion*

By Messrs. G. G. Thomas, Henry S. Jacoby, Glenn B. Woodruff, Harold C. Bird, W. A. Duff, Charles Stratton Davis, C. D. Purdon, G. E. Tebbets, J. R. Worcester, A. W. Carpenter, D. B. Steinman, W. Chase Thomson, Jonathan Jones, and P. G. Lang, Jr.

G. G. Thomas,† M. Am. Soc. C. E. (by letter).‡—The proposed specifications are of broad scope and show an intensive study by the Conference Committees of all matters involved in the live load, design, and fabrication of metal bridges. The live loading recommended (Article 203§) is an entirely new combination of axle loads and spacings and is a step toward keeping steel bridge design up with the trend and in capacity ahead of actual locomotive loadings, more especially for the majority of railroads which do not use the Mallet type. As a comparison: The Cooper E-60 equals 426 tons distributed over 112 ft.; the large Santa Fé locomotives equal 616 tons distributed over 166 ft.; and the proposeed A-64 equals 720 tons distributed over 175 ft.

It is noted in Fig. 2‡ that all the 64 000-lb. axle loads are at driving axles equally spaced. Some further consideration might well be given to this spacing with especial reference to the rear axle, which is usually a trailing truck under the firebox. It carries about the same load as a driver, and is spaced about 10 ft. from the nearest one.

* Discussion of the General Specifications for Steel Railway Bridges, continued from April, 1930, *Proceedings*.

† Engr. of Bridges, A. C. L. R. R., Wilmington, N. C.

‡ Received by the Secretary, February 14, 1930.

§ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.

The new impact formula (Article 204*) is in accord with live load tests of recent years. It will partly offset the increased weight and cost of spans less than 75 ft. long as compared with the very heavy impact increment derived from the present A. R. E. A. Specifications.

In regard to the proposed column formula (Article 301 (*a*)†), it appears to the writer that practically the same results would be obtained in the design of new bridges by the use of the formula,

$$S = 16\,000 - 70\,\frac{L}{r}$$

with a maximum allowable value of $S = 14\,000$ lb. per sq. in.

In case it is desired to use these specifications for the review of light spans, a set of unit stresses should be formulated entirely independent of unit stresses for new work, as is now the practice of the American Railway Engineering Association.‡ A formula of the Rankine type,

$$S = \frac{24\,000}{1 + \dfrac{L^2}{Z\,r^2}}$$

can be applied to such reviews of light spans with economical results for high values of $\frac{L}{r}$.

The proposed specifications, like those of the A. R. E. A., 1920, provide for a 16-ft. horizontal clearance. It is germane to compare this side clearance with double-track construction, where it is customary to build on 13-ft. track centers; yard and passing tracks are usually spaced at 13 ft., or less. The average width of railway equipment is 10 ft., to which 12 in. may be added for clearance to make a total width of 11 ft., or 5 ft. 6 in. each way, from the center of the track. For double track spaced on 13-ft. centers, there remains 7 ft. 6 in. to the center of the opposite track as compared to the 8 ft. called for in the proposed clearance diagram (Fig. 1§).

For through truss spans more than 200 ft. long the proportions of width to length permit the economical use of the proposed side clearance of 16 ft. For spans less than 200 ft. long the 16-ft. side clearance is economically wasteful and, therefore, unjustified. It is the writer's opinion that further careful consideration should be given to adopting 15 ft. for side clearance with a corresponding revision of Article 103,§ on Spacing of Trusses, Girders, and Stringers, to conform therewith, since that clearance will conform fully to present requirements and to general practice.

Article 300‖ provides for 50% increases in dead load stresses. The writer's experience is that, taking a long view of bridge design, a railroad company's interests are best served by using identical unit stresses for both dead and live loads. In such matters past experience is a good criterion for future design.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

† *Loc. cit.*, p. 2654.

‡ "Rules and Unit Stresses for Rating Existing Bridges," *Bulletin 232*, Am. Ry. Eng. Assoc., December, 1920.

§ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2650.

‖ *Loc. cit.*, p. 2653.

Devices now introduced to provide spans only sufficient for present-day traffic will result in premature replacements and attendant increased capital costs before full usefulness has been realized because, under the practically inevitable increased loadings of the future, the high unit stresses used in present design will leave no margin for continued operation. The retention of the conservative 16 000-lb. basic unit stress is a protection to railroad capital invested in steel bridges; the proposed 50% increase in stress from dead load is not.

It is obvious that no set of specifications can be assembled on such a subject that do not embody compromises and show omissions on certain points. It seems that the shop paint requirements in Article 541* are a case in point. The writer's observation of old bridges removed from service began in 1911 and has throughout covered the inspection of material in and near joints made both in the shop and in the field. Nothing observed in that time, however advantageous to bridge shop economies, can warrant the omission of paint from contact surfaces before assembly in the shop; everything observed indicates the necessity for painting contact surfaces before assembly and riveting, both in the shop and in the field. Granted that such paint is burned and hardened by the hot rivets, it nevertheless serves well to seal the edges all around the joints and to close openings which would otherwise be left open for rust.

There is no specific provision in Article 211† for the design of connections in members subject to reversal of stress. Paragraph 44 of the present A. R. E. A. Specifications provides fully for this by stating, "the connections shall be proportioned for the sum of the resultant stresses."

Article 414‡ provides for 25% splices in faced compression members. Weighing the very small added cost against the very considerable added stability, the 50% splices of the A. R. E. A. Specifications are more logical.

Fillers extending beyond the connected member should be fully developed by the rivets; Article 415‡ provides for only 50% development.

Referring to Article 428,§ the construction of girders without cover-plates up to sections requiring angles greater than 6 by 6 by $\frac{7}{8}$ in., is uneconomical in first cost and most undesirable as regards maintenance. The greater first cost (to the railroad, not to the shop) is obvious; the maintenance, unless 100% efficient, will permit rusting out of the top flange angles. A design with a full top cover-plate permits the removal of the plates without disturbing the flange angles and stiffeners, and the original flange angles are then in condition to develop a new cover-plate.

The provision in Article 428 for full bottom cover-plates is a sound innovation as providing protection to the bottom flange angles at the bridge seat, where corrosion from accumulations of dirt is likely to occur.

In closing these comments, the writer feels constrained to register his impression that, as compared to the A. R. E. A. Specifications, those proposed by the Conference Committees favor the viewpoint of the bridge shop in the

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2665.
† *Loc. cit.*, p. 2653.
‡ *Loc. cit.*, p. 2657.
§ *Loc. cit.*, p. 2659.

sale and purchase of steel bridges rather than that of the railroads. This will form a decided handicap against general adoption, in spite of the advances and improvements indicated by the specifications.

HENRY S. JACOBY,* M. AM. SOC. C. E. (by letter).†—The attempt to secure common standard specifications for steel bridges to be adopted both by the Society and by the American Railway Engineering Association merits encouragement even if it may require a number of years to complete the task through representative committees.

It is interesting to note that for many years the revision of such specifications has shown a steadily advancing tendency in the direction of increasing the application of the laws of mechanics and other theoretic considerations to the design of details that were formerly regarded as unworthy of careful thought. It used to be said that these matters were to be settled by the practical judgment of the engineer in practice. Perhaps the clearest illustration of this tendency may be discovered by comparing successive editions of the specifications for steel railway bridges issued by the American Railway Engineering Association. This is the more significant since these specifications have been the result of careful study and thorough discussion by committees representing those who design bridges for a variety of conditions on the different railroads in the United States and Canada; those who fabricate them; and those who observe their behavior in service by systematic inspection and maintenance. The discussion of the specifications in committee of the whole occurs after various groups of articles have been treated by a similar process in sub-committees. Articles 103,‡ 104,‡ 108,‡ 201,§ 214,‖ 305,§ 401,§ 403,§ and 404,¶ in the specifications now under discussion, give evidence of this tendency.

Article 103 which places each pair of stringers symmetrically under each rail is to be commended since it reduces the stresses in the ties and causes approximately equal stresses in the four stringers, whether the tie is new or old in service. Article 104 is in harmony with the increasing practice of paying more attention to deflections beyond that required for strength only.

The stipulation that spans with floor systems shall preferably have end floor-beams, without limitation of span (Article 108), is an illustration of the commendable practice of extending the use of certain features to bridges of shorter spans, after observing their value in long-span bridges. End floor-beams were first specified by consulting engineers for certain important bridges which then represented some advanced steps in construction.

Whether or not the time has arrived when it is desirable to break away from the long-established custom of specifying locomotive live loads (Article 201) of the Cooper Equivalent Loading, like E-60, E-70, etc., should preferably be discussed by bridge engineers connected with railroads. It must be

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† Received by the Secretary, February 15, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2650.

§ *Loc. cit.*, p. 2651.

‖ *Loc. cit.*, p. 2653.

¶ *Loc. cit.*, p. 2655.

remembered that the capacity of every old bridge on each railroad is evaluated in terms of the E-system, while all the locomotives in current use are evaluated in terms of the same system. Will the advantages of the proposed system of locomotive loads outweigh the disadvantages; or, to put it in a different way, may not the gain which is anticipated be more effectively obtained in some other way?

The specification (Article 204*) in which the impact is reduced two-thirds for bridges designed exclusively for electric traction is a step forward. The resulting economy in the cost of such bridges is thoroughly justified.

The requirements of Article 214 are the natural outgrowth of extended discussions on secondary stresses and are reasonable. The specification in the first sentence especially will improve the design of bridge trusses in features that need it most.

It is hoped that the Committee will present some reasons why a different unit stress is specified in Article 300† for dead load than those in Article 301, so that one may discover whether this represents a step forward or backward.

It is gratifying to observe that the slenderness ratio in Article 305 is extended to riveted tension members. The effect of such a specification as that in Article 403 on eccentric connections is more far-reaching than the primary result at which it is aimed. The continued attention of designers directed to this subject inevitably leads to the evolution of new ways to avoid eccentricity which were not previously considered to be feasible. In details on which it is impracticable to apply theory to secure better construction, some minimum is specified which is based on experience or good practice, as, for example, in Articles 405‡ and 426.§

It is desirable to discuss Articles 410‡ and 423§ at the same time. Article 410 is based chiefly, if not entirely, upon the results of experimental tests of tension members of varied compositions of plates and shapes, and it requires the use of pin-plates in construction. The relations given are to provide adequately for the curved and conflicting lines of stress in the vicinity of the pin-hole. Theoretically, the part beyond the pin-hole in each segment of the member, may be treated approximately as a beam in which the load is applied at the pin-bearing to the web and pin-plates, while the rivets in the flanges transfer a large part of the reactions from these plates to the flange angles, before the section cutting any part of the pin-hole is reached. The part referred to equals the tensile stress which the angles must carry past the pin.

Three different distributions of stress should be computed for each segment. In the sections of a segment before the pin-plates are reached the stresses in the angles and web-plate are distributed in proportion to their net sections. At the section through the center of the pin-hole a different distribution occurs, since the pin-hole cuts the web and pin-plates which, together with the angles, provide at least the specified excess of section. Behind the pin the bearing

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

† *Loc. cit.*, p. 2653.

‡ *Loc. cit.*, p. 2656.

§ *Loc. cit.*, p. 2658.

pressure of the pin is distributed in proportion to the thicknesses of bearing in the web and pin-plates.

The number of rivets which should pass through both pin-plates and web, or through pin-plates and angles, must be at least equal to those required to transfer some stresses from one to another of these parts as is required on account of these three differences in stress distribution. Practical considerations may require extra rivets to hold the parts in close contact. The second sentence of Article 423 has the effect of placing one leg of each flange angle between the web and a pin-plate, even if only one pin-plate is required in each segment. The rivets are thus in double shear and the thickness of the angles may be adjusted, if necessary, to secure a proper location of the rivets. Should not the last sentence in Article 423 be modified to include a brief but definite statement on this duty of the transfer of stress from one part to another?

In compression members which connect to a pin, as at the hip joint, it is important that one pin-plate on the outside of the angles of each segment be extended far enough until the flange rivets receive the transfer of that part of the total stress in each segment, which excludes only the stress in the web-plate as computed for any section beyond the pin-plate. Does Article 423 fully cover this requirement? The practical tendency has been to economize metal by making the pin-plates too short. A large percentage of rivets through pin-plates and web, but not through the angles, performs no function except to keep these plates in contact.

Glenn B. Woodruff,* M. Am. Soc. C. E. (by letter).†—A specification should state in as definite and in as simple rules as possible the best available theoretical and practical knowledge. Unnecessary refinement of calculation involving cumbersome formulas should not be required. It is wise to bear in mind how wide a range must be covered by the "factor of ignorance". A wide range in the physical properties of the material is permitted. Knowing these physical properties it is impossible to predict the strength of a fabricated truss member within a wide range. It is the reverse of science to express precision in a formula to a greater extent than is warranted by the data on the subject.

The use of the "simplified loadings" facilitates calculations and is to be commended. The expression for shear, however, should be stated definitely. Is this formula, ($E = 128 + \frac{1}{2} L$), to be used for web members of trusses? If used for the end post, to be consistent, it should also be used for the end chord. The same load should be used for floor-beam reactions as for moments. If the value of the concentrated load were expressed in terms of the influence line segment lengths, the results would correspond more closely with those from wheel loadings. It is the writer's opinion that the probabilities of simultaneous maximum loading are amply covered by the present A. R. E. A. Specifications.

Article 204.‡—The writer does not believe that present knowledge of impact is so definite as to warrant such a comparatively small change from the present formula. Considering that impact is to be calculated from one track

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* Cons. Engr., New York, N. Y.

† Received by the Secretary, February 15, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

only, the present formula, with its higher values for short spans, seems more conservative.

Lateral Forces.—Article 206* specifies that bracing between compression chords shall be proportioned to resist a shear of 2% of the compression in the panel. Whether or not it has a scientific basis, this Article requires bracing that is in proportion to the structure.

Article 212.†—If the intent of this Article is to make it mandatory to design verticals for the bending stresses resulting from floor-beam deflection, such requirement should be made more definite. For combined flexure and axial compression it should be permissible to design at the flexural unit stress unless the direct unit stress allowed for compression only under the axial load, is exceeded.

Article 214.†—This specification should define "other secondary stresses".

Article 301.‡—The proposed column formula seems to represent a step backward and for all practical purposes is a return to the old formula, $16\,000 - 70\,\frac{L}{r}$ (14 000 lb., maximum). No scientific basis can be claimed for any formula that has as its only variable the slenderness ratio. Knowledge of column behavior does not justify more precision than is given by the straight-line formula. The present formula, $15\,000 - 50\,\frac{L}{r}$, is more nearly parallel to the results of column tests.

Article 404.§—The requirements of one-thirtieth and one-fortieth for thicknesses of web and cover is usual, but the writer fails to see the reason for the difference. In design, the requirement for webs is generally met and that for covers is often stretched. Should not the ratio of one-fortieth apply to both webs and covers?

Article 405.‖—Quoting from the report of the Society's Special Committee on Steel Column Research,¶ "if the unsupported width of the outstanding flange is less than about 20, the flange should develop full compressive strength before buckling". Are not the values given in this Article ultra-conservative?

Article 408.‖—This Article is too complicated and the corresponding paragraph in the present A. R. E. A. Specifications is satisfactory.

*Article 414.***—The writer recommends changing 25% to read 50 per cent.

*Article 419.***—This is another case of a precise formula based on very meager data. The requirement of lacing for a shear of 2% of the axial stress is as close as is justified by the present knowledge of stress in lacing-bars.

Article 422.††—The second section of this Article does not harmonize with Article 305.§

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.
† *Loc. cit.*, p. 2653.
‡ *Loc. cit.*, p. 2654.
§ *Loc. cit.*, p. 2655.
‖ *Loc. cit.*, p. 2656.
¶ *Loc. cit.*, February, 1929, Papers and Discussions, p. 446.
** *Loc. cit.*, December, 1929, Papers and Discussions, p. 2657.
†† *Loc. cit.*, p. 2658.

*Article 444.**—It is sufficient to gear rollers to the upper plate only. This detail permits re-adjustment of the rollers if there is a movement of the masonry.

Section 5.—Workmanship.†—The writer prefers edge planing and full reaming in all material, except bracing, that carries calculated stress.

Articles 529-530.‡—In a general specification covering alloy steels, it would be consistent also to cover heat-treated eye-bars.

Part II.—Materials.§—The writer favors medium carbon steel with a yield point of 27 000 lb. per sq. in. This material can be furnished at practically no increase in cost and the unit stresses may safely be increased 15 per cent.

HAROLD C. BIRD,‖ ASSOC. M. AM. SOC. C. E. (by letter).¶—A careful perusal of these excellent specifications indicates the vast amount of study which the Conference Committees have given to the subject.

*Article 203.***—*Live Load.*—It was apparent that the Cooper loading would eventually be discarded after D. B. Steinman, M. Am. Soc. C. E., presented his paper entitled "Locomotive Loadings for Railway Bridges".†† The idea of any load system specified is not necessarily to represent an actual engine or train, but to establish an imaginary group of wheels (or its equivalent) which should give as great, if not greater, stresses than any which are likely to occur at present or in the immediate future. The new specifications give a loading which does represent a type of engine with weight on drivers and spacing that approximates modern conditions and that, in many ways, meets the requirements of actual practice.

TABLE 8.—COMPARATIVE LOCOMOTIVE SALES IN THE UNITED STATES DURING 1928 AND 1929.

Type.	Relative number of each type sold.	Maximum weight of engine, in thousands of pounds.
2-8-0	2	298
2-8-2	53	335
2-8-4	26	460
4-8-2	100	388
4-8-4	80	498
2-10-2	13	360
2-10-4	35	503
4-12-2	17	497
2-8-8-2 4-8-8-2	30	630
2-8-8-4	4	717
4-8-3-4	1	705

On a short bridge the moments and shears depend almost entirely upon the wheel system of the locomotive. For that reason consideration should be

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2661.

† *Loc. cit.*, p. 2661 *et seq.*

‡ *Loc. cit.*, p. 2664.

§ *Loc. cit.*, p. 2667 *et seq.*

‖ Chairman and Prof. Civ. Eng., Duke Univ., Durham, N. C.

¶ Received by the Secretary, February 15, 1930.

** *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.

†† *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 606.

given to the type of engines recently purchased. The relative number of each type of locomotive sold during 1928 and 1929 is shown in Table 8.*

It will probably be safe to ignore the types of locomotives which are not commonly sold depending, if necessary, on the overload clause for protection. However, any locomotive weighing more than 432 000 lb. (new A-64 loading), which has a reasonable sale, must be investigated. A brief study shows that Types 2-8-4 and 4-8-4 need not be considered. A little more study shows that Types 2-10-4 and 4-12-2 can also be ruled out.

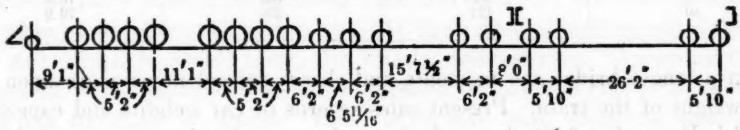

FIG. 6.

In the case of the Mallet locomotives the result is not so evident. The following calculations are based on a Mallet engine weighing 630 000 lb., with wheel spacing as shown in Fig. 6, followed by a train of 7 000 lb. per ft. In Table 9, calculations were made using both 7 000 and 6 000 lb. per ft. of train.

TABLE 9.—EQUIVALENT UNIFORM LIVE LOAD, IN THOUSANDS OF POUNDS PER FOOT OF TRACK.

For Center Moment: 630 000-Lb. Mallet Engine Followed by a Uniform Load of: 6 000 lb. per foot of train.	For Center Moment: 630 000-Lb. Mallet Engine Followed by a Uniform Load of: 7 000 lb. per foot of train.	For Center Moment: A-64 loading.	For Center Moment: Cooper's loading corresponding with A-64 loading.	Length of span, in feet.	For End Shear: 630 000-Lb. Mallet Engine Followed by a Uniform Load of: 6 000 lb. per foot of train.	For End Shear: 630 000-Lb. Mallet Engine Followed by a Uniform Load of: 7 000 lb. per foot of train.	For End Shear: A-64 loading.	For End Shear: Cooper's loading corresponding with A-64 loading.
14.00	14.00	13.20	64.0	20	16.99	16.99	16.00	64.0
10.75	10.75	12.00	73.1	40	13.34	13.34	13.50	71.6
10.84	10.84	10.70	74.3	60	12.45	12.45	11.73	71.8
10.46	10.46	9.75	72.2	80	11.50	11.50	10.57	68.1
9.74	9.74	9.13	70.7	100	10.72	10.73	9.84	65.6
9.18	9.19	8.67	67.5	120	10.12	10.18	9.63	66.5
8.69	8.73	8.33	65.8	140	9.64	9.77	9.47	67.5
8.24	8.33	8.02	64.3	160	9.25	9.45	9.26	67.9
7.88	8.00	8.00	65.8	180	8.94	9.20	9.08	68.0
7.54	7.75	8.02	67.4	200	8.68	9.00	8.91	68.2
7.10	7.50	7.91	69.5	240	8.28	8.69	8.60	68.1
6.82	7.36	7.70	70.0	280	7.99	8.46	8.36	67.9
6.63	7.27	7.48	69.5	320	7.76	8.28	8.16	67.8
6.50	7.21	7.26	68.5	360	7.57	8.14	8.01	67.7
6.40	7.17	7.09	67.6	400	7.42	8.03	7.86	67.4
6.33	7.14	6.98	67.0	440	7.30	7.94	7.75	67.3
6.28	7.12	6.88	66.5	480	7.20	7.87	7.65	67.1
6.24	7.10	6.81	66.2	520	7.10	7.79	7.56	66.9
6.21	7.08	6.75	65.7	560	7.04	7.75	7.49	66.8
6.18	7.07	6.71	65.5	600	6.96	7.70	7.43	66.7
6.10	7.04	6.57	64.9	800	6.74	7.52	7.19	66.2
6.06	7.03	6.51	64.6	1 000	6.59	7.41	7.04	65.8

The data in Tables 9, 10, and 11, indicate that the A-64 loading does not quite provide for the Mallet type of locomotive that is being sold at the present time.

* Comp. from *Railway Age*, Vol. 86 (1929), p. 76 *et seq.*; Vol. 88 (1930), p. 80 *et seq.*

TABLE 10.—COMPARISON OF FLOOR-BEAM REACTIONS.

Length of span, in feet.	Floor-Beam Reactions, in Thousands of Pounds per Track.		Cooper's equivalent loading corresponding to A-64 loading.
	Mallet engine.	A-64 type of loading.	
10	138	132	68.7
20	222	240	73.2
30	328	331	76.8
40	422	390	72.2

On a longer bridge the moments and shears depend much more upon the unit weight of the train. Present sales records of car weights and capacities would indicate that 6 000 lb. per ft. is ample, even allowing for some overload.

TABLE 11.—COMPARISON OF MAXIMUM SHEARS AND MOMENTS FOR TRUSS BRIDGES.

Number of panels.	Length of panel, in feet.	Maximum Moments, in Thousands of Foot-Pounds per Track.		Maximum Shears, in Thousands of Pounds per Track.	
		Mallet engine.	A-64 loading.	Mallet engine.	A-64 loading.
6	30	19 630	19 200	655	640
6	30	30 579	29 120	442	422
6	30	32 696	32 280	253	242
6	30	......	......	134	120
6	40	32 947	32 536	824	814
6	40	50 619	49 704	550	542
6	40	54 082	56 960	332	316
6	40	......	......	164	150
7	30	22 997	22 640	767	754
7	30	37 223	36 056	553	540
7	30	42 583	42 700	373	352
7	30	......	......	226	204
7	30	......	......	118	100
7	40	38 638	38 230	966	956
7	40	62 150	61 188	695	686
7	40	71 660	74 200	464	456
7	40	......	......	279	264
7	40	......	......	138	124
8	30	26 240	25 930	875	864
8	30	43 720	42 680	661	652
8	30	52 518	52 620	478	464
8	30	54 082	56 960	822	804
8	30	......	......	195	174
8	30	......	......	98	84
8	40	44 554	43 780	1 114	1 094
8	40	73 682	72 360	835	824
8	40	89 013	90 700	601	594
8	40	93 450	95 800	401	392
8	40	......	......	241	228
8	40	......	......	120	108

Statements have been made that the Pennsylvania System has 40-ft. gondolas, weighing 295 000 lb., or 7 400 lb. per ft. P. G. Làng, Jr., M. Am. Soc. C. E., as early as 1923, stated* that cars were then in use that weighed 8 300 lb. per ft. of track and that 7 300 lb. more nearly approached the rolling

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 670.

stock at that time. The writer feels that more data should be collected on the weight of cars per foot of track.

*Article 204.**—*Impact.*—The time is nearer at hand when designers will be able to express impact much more accurately. B. R. Leffler,† M. Am. Soc. C. E., and F. E. Turneaure,‡ M. Am. Soc. C. E., have each recently written papers dealing with the impact results of the report of the 1928 British Stress Committee. Impact depends on finding the value of the counterbalance of the locomotive wheels, its distance from the center of the driving wheel, the number of wheels on the bridge, etc. To these results must be added the effect of rail joints, irregular track, flat wheels, and swaying. Assuming that all these facts are known it becomes a question of which engine is to be adopted for the standards of design. Here, as in live load, designers must use an imaginary group of wheels that will produce impact stresses as great, if not greater than, any that are likely to occur at present or in the immediate future.

The Conference Committees, apparently realizing that it may be many years before the results of these tests can be used in a practical manner, have developed a formula which seems to fit the results of past experience. The new formula is simple to use and closely resembles that§ of Henry B. Seaman, M. Am. Soc. C. E., when his points of tangency are changed from 1 000 to 800 ft. and from 125 to 100 per cent. It gives almost identical results between 100 and 600 ft.

Article 301.‖—*Compression Formula.*—In accordance with the comments so frequently made, the Rankine formula should undoubtedly be adopted. Would it not be desirable to authorize the optional use of a straight line formula, such as $16\,000 - 70\,\frac{L}{r}$, with its proper maximum value and slenderness ratio limit?

Article 301.‖—*Compression Flanges of Plate Girders.*—The writer would like to know just why this formula has been changed. The use of the constant, 2 000, gives unit stresses, through the ordinary range of a plate girder, 450 to 550 lb. higher than the old A. R. E. A. formula.

Article 419.¶—*Latticing.*—The formulas proposed are in a form which should receive commendation. One might possibly question using a ratio of secondary stress of 20% in Equation (2).

*Article 433.***—*Intermediate Stiffeners.*—An inspection of the proposed formula shows that the constant in Rankine's formula is $\frac{1}{240\,000}$ and gives results identical with the old formula when the unit shear is 8 000 lb. When the unit shear is 5 000 lb. the new formula requires the stiffeners to be about

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

† "A Study of the Causes of Impact on Railroad Bridges," *Bulletin 218*, A. R. E. A. August, 1929, p. 1.

‡ *Bulletin*, A. R. E. A., September, 1929, p. 5.

§ *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 354.

‖ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2654.

¶ *Loc. cit.*, p. 2657.

** *Loc. cit.*, p. 2659.

10 in. closer together, and at 10 000 lb., about 10 in. farther apart, than with the old formula.

Welding.—A supplement to these specifications should treat the subject of welding. The Society should be the pioneer in this movement. In fields other than bridge work, great progress has been made, as might be illustrated by the fact that in 1923 a riveted range boiler was a curiosity, because riveting of boiler seams had been supplanted by welding.

W. A. DUFF,* ESQ. (by letter).†—The following comments and suggestions are submitted as discussion of the General Specifications for Steel Railway Bridges.

Article 2.‡—*Proposals.*—Change the second sentence to read:

"The proposal shall, unless otherwise directed or allowed by letter of invitation, be based on plans and specifications furnished by the Company."

Article 3.‡—*Shop Drawings.*—The first sentence of the second paragraph should be modified to read:

"Shop drawings shall be made on the dull side of the tracing cloth, and, unless otherwise specified by the Engineer, shall be 24 by 36 in. in size, including margins".

Article 104.§—It should be made quite clear whether the increase in section to keep down deflection, as specified in the last sentence, is intended to apply to shallow trusses and plate girders, or only to rolled beams.

Article 203.‖—A clause should be added to take care of solid floors, as follows:

"The live load for proportioning transverse beams supporting ballasted decks shall be taken as a uniform load per linear foot of track equal to 22% of the maximum specified axle load. For proportioning transverse beams supporting track rails, either direct or through steel deck plates, the uniform live load per linear foot of track shall be taken as 33⅓% of the maximum specified axle load. For ballasted decks the live load shall be considered as uniformly distributed over a width of 10 ft."

The value of 22% for ballast deck is arrived at by assuming that if transverse beams are spaced 18 in. from center to center the maximum live load on any single beam will not exceed one-third the maximum axle load.

Article 210.¶—As the maximum longitudinal force may occur from the application of the brakes throughout the length of the train, a logical formula would seem to be one in which the percentage of longitudinal force decreased, with an increase in the length contributing to the force. A formula of this type is:

$$T = \frac{3}{4}(4 - L)^2 + 10 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, T is the longitudinal force as a percentage of live load, and L is the loaded length, in units of 100 ft.

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† Received by the Secretary, February 19, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2649.

§ *Loc. cit.*, p. 2650.

‖ *Loc. cit.*, p. 2651.

¶ *Loc. cit.*, p. 2653.

*Article 211.**—As this clause reads, the entire dead load stress may be taken as balancing an equal amount of live load stress of an opposite kind. This may result in bridges with excessive vibration, particularly in pin-connected trusses. Some fraction, say, two-thirds, of the dead load stress, should be allowed to offset a live load stress of opposite sign.

*Article 214.**—An increase in the allowable unit stresses might be allowed for a combination of secondary and direct stresses.

Article 301.†—The clause introducing the formula for allowable compression in flanges of plate girders should read, "compression flanges of plate girders and rolled beams."

As the use of rivets in tension is sometimes unavoidable and as rivets undoubtedly have a safe tensile unit stress, some allowable unit stress should be specified with a clause to the effect that they are to be used in tension only when unavoidable and with the permission of the Engineer.

The clause requiring a reduction of 25% in the allowable unit stress in floor connection rivets should be retained to take care of tension and other unusual stresses to which these connections may be subjected.

Article 305.‡—As vertical members have less tendency toward lateral deflection than horizontal members, a higher ratio of $\frac{L}{r}$ can be allowed in vertical members. The writer suggests the following:

"The radius of gyration shall not be less than that given by the following formulas:

"For main compression members:

$$r = \frac{2L + H}{300} \qquad (4)$$

"For wind bracing and other secondary compression members:

$$r = \frac{2L + H}{400} \qquad (5)$$

in which,

r = radius of gyration, in inches.
L = unsupported length of members, in inches.
H = the projection of the unsupported length on a horizontal plane, in inches. This factor is equal to zero for any axis in which there is no bending stress in the member due to its own weight.

"For built-up I-sections the radius of gyration may be computed for the flange materials only, in which case only the flange materials may be counted on as effective section for carrying axial stress".

Article 404.‡—For practical reasons, it sometimes is desirable to allow material of an unsupported width exceeding forty times its thickness at a reduced stress and the writer suggests the following:

"No material used in compression shall have an unsupported width of more than sixty times its thickness, and not more than forty times the thickness of the unsupported material shall be considered as effective section."

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.
† *Loc. cit.*, p. 2654.
‡ *Loc. cit.*, p. 2655.

*Article 435.**—Top lateral bracing should be used in long stringer spans and while stringers might be considered as deck spans the subject of stringers should be included in the last paragraph of this clause, in order that they may not be overlooked.

Article 504.†—Sub-punching and reaming after assembly should be done to remove material injured in punching, to insure smooth, even holes with rivets of full cross-section and to facilitate the driving out of rivets, when necessary.

CHARLES STRATTON DAVIS,‡ M. AM. SOC. C. E. (by letter).§—These General Specifications for Steel Railway Bridges, are excellent and bear evidence of much study in their preparation. The writer, however, offers the following constructive criticisms and suggestions.

In Article 209‖ provision is made for stresses due to sway of engine; the force specified is fixed at 20 000 lb. regardless of the live load adopted. It seems to the writer that this force should vary with the live load, as provided for in Article 203¶; it could well be made proportional to the live load adopted.

In Article 210‖ provision is made for the longitudinal force to be applied 6 ft. above the tops of rails. Since the longitudinal force is applied to any structure through frictional resistance between the wheels and the rails, it would seem that this force should be applied at the tops of the rails.

In Article 300‖ the note in parenthesis provides for increasing live load and impact stresses 50%, adding these to the dead load stresses, and then proportioning sections with the specified unit stresses increased 50 per cent. It would be simpler in actual performance of the operation to add two-thirds of the dead load stress to the other stresses and use the unit stresses as specified. (See Article 433,** where this is done.) It seems that wind and sway stresses are provided for at live load units and not dead load units. This should be made clear.

In Article 431,** it would be better to fix the distance over which a wheel load should be distributed at some definite amount as, say, 36 in. The size and spacing of ties may vary; therefore, the value, as written, becomes indefinite.

In Article 433 it would be better to base the width of the intermediate stiffeners upon the size of the rivets used rather than upon the depth of the girders. The writer suggests the use of 3 in. for ¾-in. rivets, 3½ in. for ⅞-in. rivets, and 4 in. for 1-in. rivets.

Article 443†† might well be enlarged to provide for an inlay bronze plate in the lower of the two sliding plates; this plate to be about ⅝ in. thick. This would call for a new section covering the manufacture of bronze.

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* *Proceedings,* Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2660.
† *Loc. cit.,* p. 2662.
‡ Cons. Engr., Pittsburgh, Pa.
§ Received by the Secretary, February 21, 1930.
‖ *Proceedings,* Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.
¶ *Loc. cit.,* p. 2651.
** *Loc. cit.,* p. 2659.
†† *Loc. cit.,* p. 2661.

Article 448* provides for cambering trusses; provisions should also be made for cambering plate girders. Because it is not a complete statement, the writer believes it would be better to omit the clause "ordinarily, this will be effected by increasing the length of the top chord ⅛ in. for each 10 ft."

In Article 541† the writer would suggest adding the clause: "All contact surfaces shall be thoroughly cleaned before assembling."

In Article 701‡ it is suggested that the last sentence be made to read: "Any weight in excess of 1½% more than the computed weight of any member shall not be included."

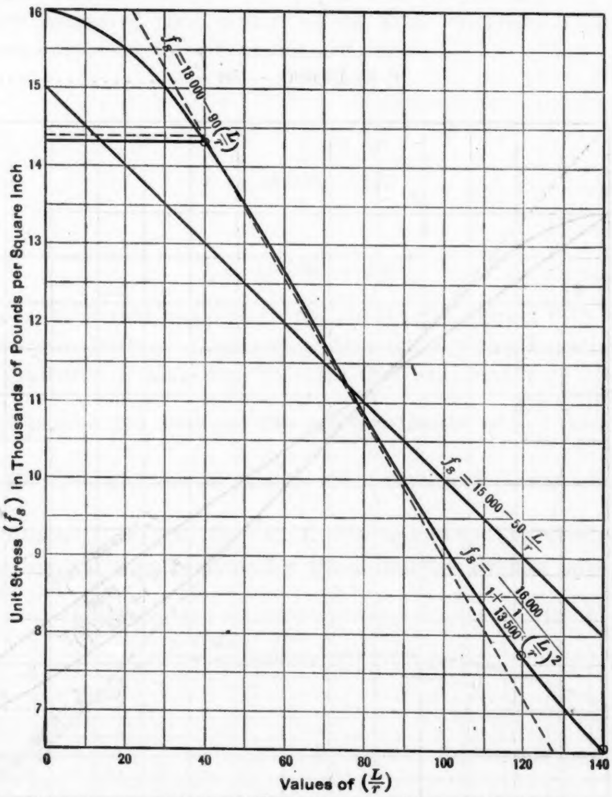

FIG. 7.—COMPARISON OF FORMULAS FOR AXIAL COMPRESSION.

The formulas adopted for unit stresses in members in axial compression (Article 301) may be scientifically derived, and conform more nearly to a mathematical analysis of the data available than one of a simpler form; but, in view of the fact that steel sections cannot be rolled to exact dimensions (under Article 826§ a variation of 2½% larger or smaller is permissible) and also that neither the yield point nor the ultimate strength can be fixed

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2661.

† *Loc. cit.*, p. 2665.

‡ *Loc. cit.*, p. 2666.

§ *Loc. cit.*, p. 2669.

with absolute accuracy, the writer believes that the formula suggested by the Committee makes calculations unnecessarily laborious. Fig. 7 illustrates the variations in three formulas suggested for axial compression, gross section:

$$f_s = \frac{16\ 000}{1 + \dfrac{1}{13\ 500}\left(\dfrac{L}{r}\right)^2} \quad \text{.......................(6)}$$

$$f_s = 18\ 000 - 90\,\frac{L}{r}\ \text{(maximum, 14 400 lb.)} \quad \text{.............(7)}$$

and,

$$f_s = 15\ 000 - 50\,\frac{L}{r} \quad \text{.......................(8)}$$

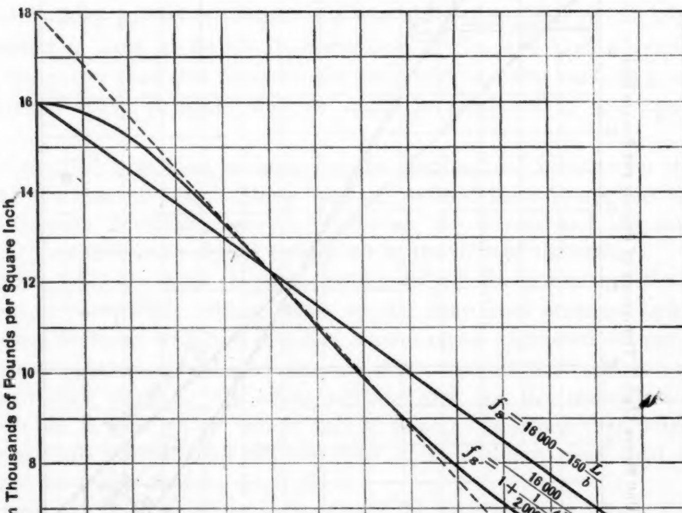

FIG. 8.—COMPARISON OF FORMULAS FOR COMPRESSION IN THE FLANGES OF BEAMS.

Equation (6) is that adopted in Article 301;* Equation (8) is the one adopted by the American Railway Engineering Association;† and Equation (7) is a proposed substitute straight-line formula that conforms closely to

* Taken from *Bulletin 307*, A. R. E. A.

† *Proceedings*, A. R. E. A., Vol. 25, 1924.

Equation (6) between the limits fixed in Articles 301* and 305,† that is, between $\frac{L}{r} = 40$ and $\frac{L}{r} = 100$ for main compression members.

The greatest variation is about 2.2% which occurs when $\frac{L}{r} = 100$. Equation (7) is in line with good practice in that it gives slightly larger sections for the most slender columns. Equation (6) proposed by the Joint Committee, brings to mind the expression, "infinite minutia."

The same criticism applies to the formula adopted for unit stresses in compression flanges of plate girders as for axial compression (Article 301). Fig. 8 shows curves for three formulas as follows:

$$f_s = \frac{16\,000}{1 + \frac{1}{2\,000}\left(\frac{L}{b}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$f_s = 16\,000 - 150\,\frac{L}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

and,

$$f_s = 18\,000 - 230\,\frac{L}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Equation (9) is that proposed in Article 301;‡ Equation (10) was adopted by the American Railway Engineering Association,§ and Equation (11) is a proposed substitute straight-line formula that conforms very closely to the curve of Equation (9) between the common limits of $\frac{L}{b}$; that is, $\frac{L}{b} = 12$ and $\frac{L}{b} = 40$. The greatest variation is about 2% which occurs when $\frac{L}{b} = 12$.

C. D. Purdon,‖ M. Am. Soc. C. E. (by letter).¶—The Conference Committees on General Specifications for Steel Railway Bridges have proposed a

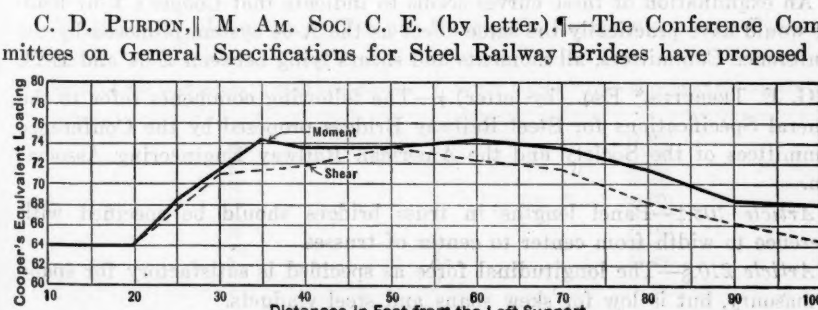

Fig 9.—Comparison of Maximum Moments and Shears on Beams or Girders Due to the A-64 Loading in Terms of Equivalent Cooper Loading.

new standard loading for bridges (Article 203**). It has been claimed by some engineers that the Cooper system of loading did not correspond closely enough

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2654.

† *Loc. cit.*, p. 2655.

‡ Taken from *Bulletin 307*, A. R. E. A.

§ *Proceedings*, A. R. E. A., Vol. 25, 1924.

‖ Cons. and Valuation Engr., St. L. S. W. Ry., St. Louis, Mo.

¶ Received by the Secretary, February 28, 1930.

** *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.

with actual locomotives. The same criticism might well be applied to the A-64 loading recommended by the Committees.

The similarity of the two systems is illustrated in Figs. 9 and 10. For example, on a 20-ft. beam or girder (Fig. 9), the same moment and shear produced by an A-64 engine would also be produced by the Cooper E-64 loading. For a 60-ft. beam or girder the same maximum moment produced by the A-64 engine would be produced by Cooper's E-74; the maximum shear would be produced by an E-72 loading. Fig. 10 illustrates a similar relationship for truss bridges of various spans with panels uniformly 25 ft. long.

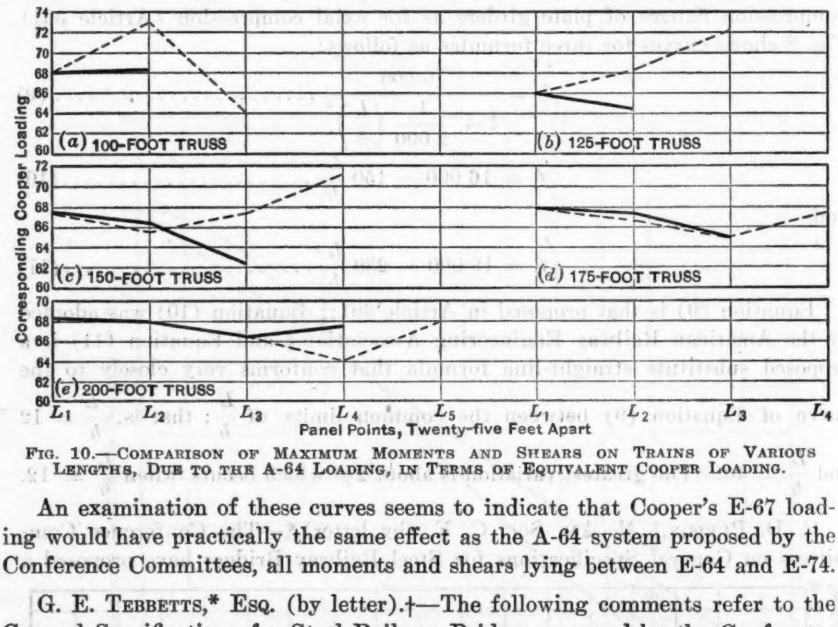

FIG. 10.—COMPARISON OF MAXIMUM MOMENTS AND SHEARS ON TRAINS OF VARIOUS LENGTHS, DUE TO THE A-64 LOADING, IN TERMS OF EQUIVALENT COOPER LOADING.

An examination of these curves seems to indicate that Cooper's E-67 loading would have practically the same effect as the A-64 system proposed by the Conference Committees, all moments and shears lying between E-64 and E-74.

G. E. TEBBETTS,* ESQ. (by letter).†—The following comments refer to the General Specifications for Steel Railway Bridges proposed by the Conference Committees of the Society and the American Railway Engineering Association.

Article 103.‡—Panel lengths in truss bridges should be specified with reference to width from center to center of trusses.

Article 210.§—The longitudinal force as specified is satisfactory for spans on masonry, but is low for skew spans and steel viaducts.

Article 214.§—The subject of secondary stresses should be treated more fully for cases in which the width of members is more than one-tenth the length. In this Article, the increase in unit stresses necessary to comply with Article 301 should be specified more plainly.

Article 301.‖—A dead load stress of 24 000 lb. per sq. in. seems to be rather high.

* Engr. of Structures, Chic. Rapid Transit Co., Chicago, Ill.

† Received by the Secretary, February 28, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2650.

§ *Loc. cit.*, p. 2653.

‖ *Loc. cit.*, p. 2654.

*Article 401.**—The thickness of web plates for plate-girder spans should be specified. Metal, $\frac{5}{16}$ in. thick, is light, except for unimportant members.

Article 442.†—A minimum of 400 ft. is rather long for spans before provision for expansion in the floor is made.

It seems to the writer that the specification will give rather light details. This is a mistake because failure occurs more often in details than in main members.

J. R. Worcester,‡ M. Am. Soc. C. E. (by letter).§—The suggestion in Article 300‖ is somewhat revolutionary and will no doubt require time before meeting general acceptance. It is not only somewhat startling, after so many years of almost universal use of a basic tensile unit of 16 000 lb. per sq. in., to have this unit increased by a considerable percentage, without any increase in the required strength of the steel, but it is also a little disconcerting to see a recommendation that the dead load stresses may be assumed as two-thirds what they actually are; for that is the effect of the paragraph. Then, again, looking at it another way, to allow 24 000 lb. per sq. in. in tension, 21 450 lb. in compression, or 36 000 lb. in bearing, for a static, constant stress in either case seems to indicate a perfection of workmanship, a refinement of calculation, and a permanence of the protective covering beyond what most engineers claim to have attained.

To be sure, this method of compensating for the effect of inertia in a structure is logical, and furnishes an easy method of accomplishing a result well recognized as desirable, but under previous specifications not attainable. To this extent it is attractive. It should be understood, however, that in its present form it involves an appreciable decrease in the factor of safety. Table 12 indicates this decrease.

TABLE 12.—Increase of Tensile Unit with Increase of Dead Stress.

Ratio of live load stress to dead load stress.	Actual tensile stress, in pounds per square inch.
∞	16 000
14/8	17 000
2	18 000
10/9	19 000
2/3	20 000
2/5	21 000
2/9	22 000
2/21	23 000
0	24 000

There is one other objection to Article 300, possibly more sentimental than real; that is, in its wording it loses sight of the rule advocated, if not originated, by the late C. C. Schneider, Past-President, Am. Soc. C. E., more than a generation ago, to reduce all stresses to the static equivalent, and to this

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2655.

† *Loc. cit.*, p. 2661.

‡ Cons. Engr., Boston, Mass.

§ Received by the Secretary, March 10, 1930.

‖ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.

equivalent static stress to apply units which may be determined experimentally to be safe and proper. By a modification of the arrangement in the proposed specification this same method of dealing with stresses might be attained. The increment in the live stress, suggested in the parentheses of Article 300, might be incorporated in the provision (Article 204*) for impact.

The live stress including impact is:

$$L' = L\left(1 + \frac{400 - \frac{l}{2}}{400 + l}\right)$$

in which, L' is the increased live stress; L is the original live stress; and l is the loaded length.

If this stress is increased by 50%,

$$L'' = \frac{3}{2}\,L' = L\left(1 + \frac{800 - \frac{l}{4}}{400 + l}\right)$$

in which, the last term indicates the necessary allowance for impact.

If Article 204 were thus stated, the sum of dead, live, and impact stresses would give the static equivalent, and the table of units in Article 301† should be modified to what are considered the safe stresses for static application. Whether these should be based on a tensile stress of 16 000, 18 000, or 20 000, or 24 000 lb. per sq. in. might then be discussed with a clearer understanding.

A. W. CARPENTER,‡ M. AM. SOC. C. E. (by letter).§—These specifications follow the general plan of the preceding specifications in all fundamentals. The arrangement of a locomotive wheel load (Fig. 2‖), an alternative uniform live load with one movable concentration (Article 203¶), and a new impact formula with reductions for girders and trusses carrying multiple tracks (Article 204**), seem to be the principal new features. There are also some minor new features, such as the rules for computing net section through rivet holes (Article 408‖), and formulas for latticing (Article 418††).

The wheel loading represents modern locomotive forms better than the Cooper system and seems to be satisfactory.

The impact formula is a simple one to apply. It gives percentages of live load varying from about 96% to 75% for spans of 10 to 80 ft., as against 117% to 80% by the formula of the 1923 Am. Soc. C. E. Specifications,‡‡ while for spans of from 80 to 400 ft., there is not a great difference. The A. R. E. A. Specification formula gives greater percentages for spans up to about 140 ft. and less for spans greater than 140 ft. The new formula seems to give rea-

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.
† *Loc. cit.*, p. 2654.
‡ Asst. Valuation Engr., N. Y. C. Lines, New York, N. Y.
§ Received by the Secretary, March 19, 1930.
‖ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.
¶ *Loc. cit.*, p. 2652.
** *Loc. cit.*, p. 2656.
†† *Loc. cit.*, p. 2657.
‡‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 478.

sonable results for single-track spans and probably is as good as can be provided until further knowledge of impact is available.

The impact is to be applied to the full live load on one track only regardless of the number of tracks carried by the bridge. This means that the assignment of total impact loads and stresses to the center truss of a four-track, three-truss bridge is the same as that to the outside trusses. In the period not so long ago when bridge specifications called for a live load stress of one-half the dead load stress (which was equivalent to a uniform allowance of 100% for impact) a center truss would have been assigned the equivalent of four full-track live loads at dead load stress (2 static track loads, doubled for impact). Under these new specifications the assignment for live load and impact on a 200-ft. span would be 2.27 full-track static live loads, or less than 57% by the old specifications; with the increased dead load stress permitted by these new specifications, it is likely that the carrying capacity in static load would be nearer 50 per cent. The old specifications were far too severe, of course, but engineers should be careful not to go too far the other way. Experimental data on impact on multiple-track bridges are scarce and appear to be needed.

The writer regrets to note what appears to him a step backward toward the Rankine form of column formula (Article 301*).

Certain minor features and requirements may be commented upon in the several following paragraphs.

The girders of deck spans and stringers of through spans must be spaced closer than 6 ft. 6 in. between centers, if more than one girder or stringer per rail is used under the requirement of Article 303.†

Why specify that "spans with floor systems preferably shall have end floor-beams" (Article 108‡), when such beams are impractical structurally with skew bridges? If used with square end bridges they should be kept well away from back-walls to provide access for cleaning and painting.

In Article 301(*c*), "Wooden Cross-Ties,"† the allowable tension stress in extreme fibers specified for white oak and dense yellow pine is 50% more than that for dense Douglas fir. The working stresses for dense Southern pine and dense Douglas fir, adopted in 1929 by the A. R. E. A. for wooden bridges and trestles, and reported as recommended by the U. S. Forest Products Laboratory, are practically the same for each. It may be that there is some difference in the strength of dense Douglas fir grown in different localities, and it is not quite clear that the A. R. E. A. classification is not confined to the fir grown in the Pacific Coast Region. The consideration given by the Committees, resulting in the assignment of working stresses to these timbers, would seem to be a matter of interest.

The minimum thickness of $\frac{3}{8}$ in. for railroad bridge material has long been standard, yet in Article 401† it is reduced to $\frac{5}{16}$ in. This seems another step backward. Main gusset-plates for trusses might well be held to a minimum thickness of $\frac{1}{2}$ in., instead of $\frac{3}{8}$ in. as specified.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2654.
† *Loc. cit.*, p. 2655.
‡ *Loc. cit.*, p. 2650.

On the subject of workmanship, with respect to the production of rivet holes, it is probable that the specification will not satisfy many. This is a subject that probably has not been given the amount of study that its importance warrants. If some of the effort devoted to such subjects as engine diagrams and column formulas were transferred to this subject, some more practical benefits might result.

With reference to Part II, "Materials",* the specifications for structural steel are stated in a footnote to be "in substantial agreement with A. S. T. M. specifications—Serial Designation A7-24". It would perhaps be more adequate to state that they are "substantially identical". The A. S. T. M. specifications A7-24 have been revised, principally to provide for copper steel when desired by the purchaser, the new issue bearing the serial designation, A7-29. It is suggested that the new issue be substituted in the specifications under consideration.

No acknowledgment is made of the A. S. T. M. origin of the silicon steel, nickel steel, and cast steel specifications. The two former are substantially and respectively the A. S. T. M. Standard Specifications for Structural Silicon Steel, A94-29, and for Structural Nickel Steel, A8-29. The cast steel specifications follow the A. R. E. A. adaptation, with modifications, of the A. S. T. M. Standard Specifications for Steel Castings A27-24. It is hereby suggested that due acknowledgments be shown for these specifications.

On this subject of the use of A. S. T. M. specifications for materials, it is suggested that some co-operative arrangement might be provided, whereby A. S. T. M. revisions might be specified to be followed after proper endorsement.

In concluding, it is noted that the specifications contain no section on erection or field assembly, which would seem to be desirable to make them cover the completed structures.

D. B. STEINMAN,† M. AM. SOC. C. E. (by letter).‡—Two aspects of the proposed new specification for live load for railway bridges are distinctly gratifying:

1.—It represents a final admission that the old standard Cooper loading does not meet modern requirements and that the time has come to adopt a new standard loading.

2.—It recognizes the admissibility of a simplified loading to be specified for the computation of stresses.

While the proposed new specification records these elements of progressive thought on the subject, the particular loading that it proposes is distinctly disappointing.

The outstanding reason for seeking a new specification to supersede Cooper's E-system is to secure a loading that will be adequately high for short spans without being too high for long spans. Ample sections for short spans (and floor systems) and economy for long spans are the governing desiderata. The system of M-loading (see Fig. 11), adopted in 1923 by the

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2667.

† Cons. Engr. (Robinson & Steinman), New York, N. Y.

‡ Received by the Secretary, March 20, 1930.

Society's Special Committee on Specifications for Bridge Design and Construction,* fully met these requirements; the proposed new A-64 loadings† fail to satisfy them.

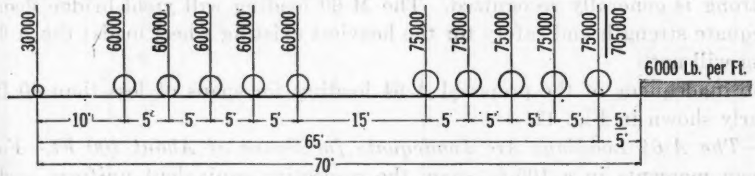

FIG. 11.—M-60 LOADING.

Objections to the Proposed A-64 Loadings.—The outstanding objections to the proposed A-64 loadings are that:

1.—They are grossly inadequate for short spans;
2.—They are likewise inadequate for spans of about 100 ft.;
3.—They are excessive for longer spans;
4.—The proposed wheel-load diagram (Fig. 2†) is unnecessarily long and complicated;
5.—The proposed diagram is defective as a pictorial representation of existing locomotives;
6.—The A-64 loading is lacking in desired simplicity and convenience;
7.—For shears, the proposed loading is unscientific and yields incorrect results for certain stresses; and
8.—The A-64 loading and its simplified equivalents do not agree closely in stress-producing effects.

These objections will be considered individually and discussed in more detail.

1.—The A-64 Loadings Are Grossly Inadequate for Short Spans.—For spans of less than 40 ft., the A-64 loadings (both wheel diagram and simplified loading) specify maximum axle concentrations of 64 000 lb. This is decidedly inadequate to represent the stress-producing effects of modern engines. Actual present-day locomotive axle loads are as listed in Table 13.

TABLE 13.—COMPARISON OF AXLE LOADS ON MODERN LOCOMOTIVES.

Engine.	Type.	Axle loads, in pounds.
Virginian Electric	2- 8-2	79 000
Pennsylvania Railroad	2-10-0	78 000
Norfolk and Western Electric	2- 8-2	75 000
Chicago, Cincinnati, Cleveland and St. Louis Railway	2- 8-2	74 400
Denver and Rio Grande Western Railroad	4- 8-2	73 000
Chicago, Burlington and Quincy Railway	2-10-4	72 800
Pennsylvania Railroad	2-10-0	72 600

Existing engines produce stresses materially higher than those given by the A-64 loadings, as follows:

For 5-ft. spans....................23% higher
For 10-ft. spans....................23% higher
For 15-ft. spans....................17% higher
For 20-ft. spans....................15% higher
For 25-ft. spans....................10% higher

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 477.
† *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.

The foregoing serious objection does not apply to the M-60 loading, which specifies maximum axle concentrations of 75 000 lb.

The importance of making the floors of railway bridges adequately heavy and strong is generally recognized. The M-60 loading will yield bridge floors of adequate strength and safety for the heaviest existing wheel loads; the A-64 loading will not.

The inadequacy of the proposed A-64 loading for spans of less than 40 ft. is clearly shown in Fig. 12.

2.—The A-64 Loadings Are Inadequate for Spans of About 100 Ft.—For mid-span moments in a 100-ft. span, the respective equivalent uniform loads are, as follows:

	Pounds per foot.
Heaviest existing locomotives	9 850
M-60 loading	9 930
A-64 loading diagram	9 130
A-64 simplified loading	8 960

Hence, the A-64 loadings are inadequate.

For end shears in a 100-ft. span, the respective equivalent uniform loads are, as follows:

	Pounds per foot.
Heaviest existing locomotives	10 750
M-60 loading	10 480
A-64 loading diagram	9 840
A-64 simplified loading	9 960

Again, the A-64 loadings are inadequate.

The "heaviest existing locomotives" in the foregoing comparisons are those considered and tabulated by the Conference Committees as a basis for investigating and comparing different proposed loadings. These comparisons show that the A-64 loadings are deficient at span lengths of about 100 ft. In this span range, existing locomotives produce stresses 8 to 10% higher than the A-64 loadings. (See Fig. 12.) The M-60 loading does not suffer from this defect. (See Fig. 13.)

When the equivalent uniform loads for the A-64 loading diagram are plotted, the resulting graph shows an irregularity in the form of a depression or "air-pocket" at or near a span length of 100 ft. (See Fig. 14.) Cooper's loading yielded a similar "air-pocket". This defect is associated with the use of a double-header locomotive for a loading diagram, and represents a resulting design weakness of the affected spans to carry locomotives of other than the assumed length and wheel-spacing. The M-loading was devised so as to eliminate this "air-pocket" defect. (See Fig. 13.)

The proposed simplified A-64 loading is of such form (a uniform load plus a designated concentration) that it cannot be adjusted to yield adequate results for span lengths of about 100 ft. without yielding excessive results for shorter spans or longer spans, or both. The simplified loading specification proposed by the Committees (for spans of more than 40 ft.) is exactly equivalent to the formulas:

$$U_M = 6\,400 + \frac{256\,000}{L} \quad \text{.....................(12)}$$

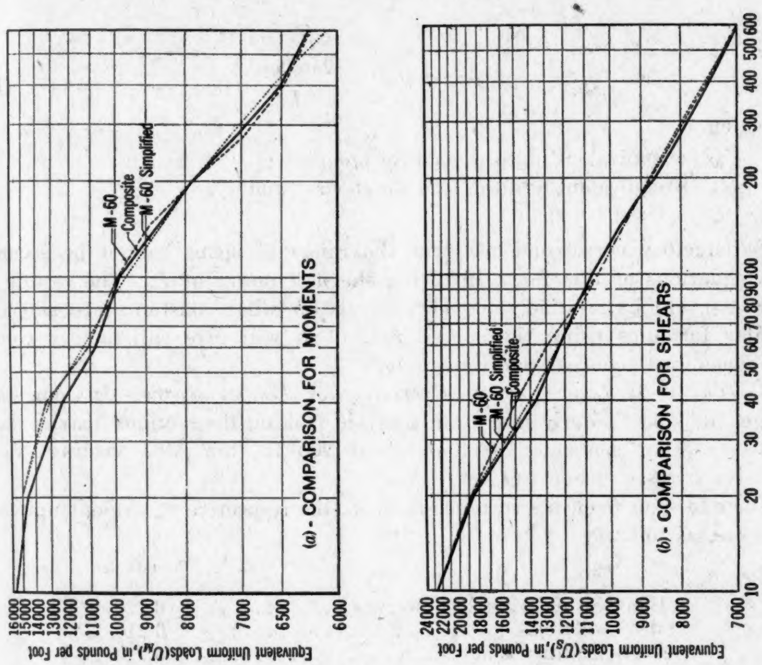

FIG. 13.—COMPARISON OF M-LOADINGS WITH COMPOSITE OF HEAVIEST LOCOMOTIVES.

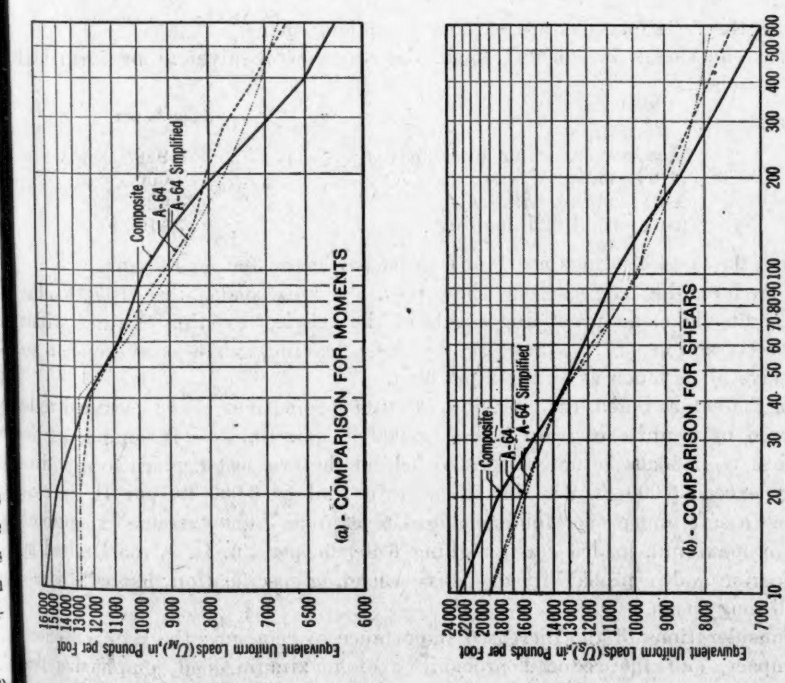

FIG. 12.—COMPARISON OF A-64 LOADINGS WITH COMPOSITE OF HEAVIEST LOCOMOTIVES.

and,

$$U_S = 7\,400 + \frac{256\,000}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which,

U_M = equivalent uniform load for moments;
U_S = equivalent uniform load for shears; and
L = length of span.

Satisfactory correspondence over the range of spans cannot be secured with equations of this form, involving the first power of L. The results at $L = 100$ will be relatively too low, no matter what constants are adopted. Similar formulas using the square root of L will give satisfactory correspondence for the entire span range.

3.—The A-64 Loadings Are Excessive for Longer Spans.—In long-span bridges, it is particularly important to avoid making the sections heavier than necessary. For spans longer than about 200 ft., the A-64 loadings yield excessive stresses. (See Fig. 12.)

For mid-span moments in a 600-ft. span, the respective equivalent uniform loads are, as follows:

	Pounds per foot.
Heaviest existing locomotives	6 220
M-60 loading	6 210
A-64 loading diagram...................	6 710
A-64 simplified loading.................	6 810

Hence, the A-64 loadings are too heavy for long spans.

For end shears in a 600-ft. span, the respective equivalent uniform loads are, as follows:

	Pounds per foot.
Heaviest existing locomotives	6 980
M-60 loading	7 000
A-64 loading diagram....................	7 420
A-64 simplified loading................	7 810

Again, the A-64 loadings are found to be too heavy for long spans.

The foregoing comparisons show how, for long spans, the M-60 loading closely fits the stress-producing effects of the heaviest existing engines within a small fraction of 1%, whereas the A-64 loadings yield stresses that are excessive by as much as 10 or 12 per cent.

In the values listed, the "heaviest existing locomotives" were assumed to be followed by a uniform train load of 6 000 lb. per lin. ft. Inasmuch as the heaviest train loads actually operated behind the heaviest modern locomotives rarely exceed 5 000 lb. per lin. ft., a train load of 6 000 lb. per ft. is considered a safe and proper following load to assume. The extreme exceptional case of maximum loaded cars weighing 6 400 lb. per lin. ft. is too limited in application and in probability to justify a loading specification that will penalize all long spans.

Considerations of the increased importance of economy, the reduced effect of impact, and the reduced probability of maximum load, emphasize the

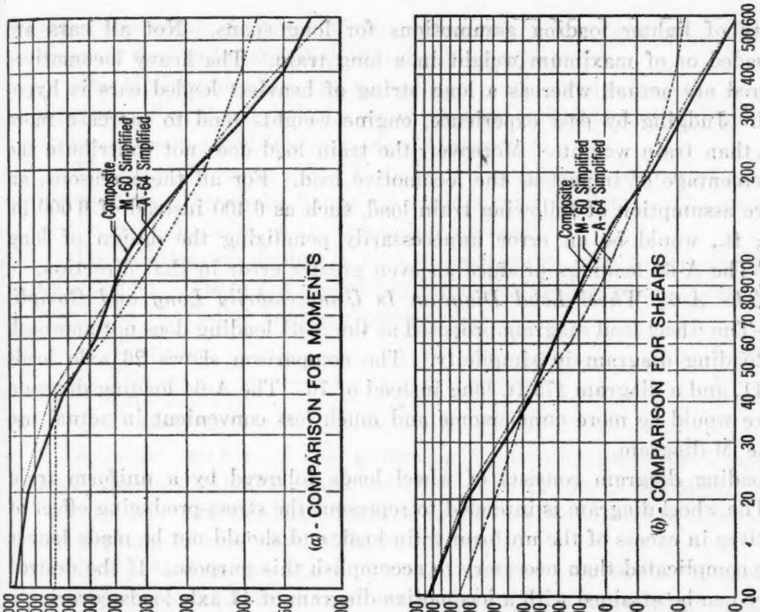

FIG. 15.—COMPARISON OF SIMPLIFIED LOADINGS WITH COMPOSITE OF HEAVIEST LOCOMOTIVES.

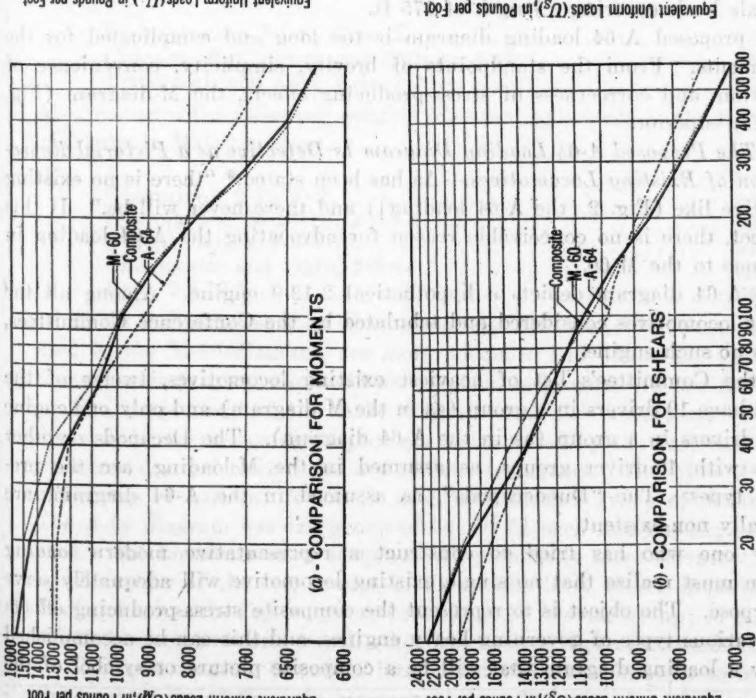

FIG. 14.—COMPARISON OF A-64 AND M-60 LOADINGS WITH COMPOSITE OF HEAVIEST LOCOMOTIVES.

propriety of lighter loading assumptions for long spans. Not all cars are fully loaded or of maximum weight in a long train. The heavy locomotives considered are actual, whereas a long string of heaviest loaded cars is hypothetical. Judging by past experience, engine weights tend to increase more rapidly than train weights. Moreover, the train load does not contribute the same percentage of impact as the locomotive load. For all these reasons, an excessive assumption of following train load, such as 6 400 instead of 6 000 lb. per lin. ft., would be an error unnecessarily penalizing the design of long spans. The A-64 loadings produce an even greater error in that direction.

4.—The A-64 Wheel Load Diagram Is Unnecessarily Long and Complicated.—The wheel load diagram proposed as the A-64 loading does not approach the M-loading diagram in simplicity. The comparison shows 26 axle loads *versus* 11, and a diagram 175 ft. long instead of 70. The A-64 loading diagram therefore would be more cumbersome and much less convenient in actual use than the M-diagram.

A loading diagram consists of wheel loads followed by a uniform train load. The wheel diagram is intended to represent the stress-producing effect of locomotives in excess of the uniform train load, and should not be made longer or more complicated than necessary to accomplish this purpose. If the desired objective can be attained with a locomotive diagram of 11 axle loads covering a length of 70 ft., it is illogical and unscientific to advocate a loading diagram of 26 axle loads covering a length of 175 ft.

The proposed A-64 loading diagram is too long and complicated for the requirements. From the standpoints of brevity, simplicity, convenience of application, and correctness of stress-producing effects, the M-diagram (Fig. 11) is far superior.

5.—The Proposed A-64 Loading Diagram Is Defective as a Pictorial Representation of Existing Locomotives.—As has been stated,* "there is no existing locomotive like (Fig. 2 [the A-64 loading]) and there never will be." If this is correct, there is no conceivable reason for advocating the A-64 loading in preference to the M-60.

The A-64 diagram depicts a hypothetical 2-12-0 engine. Among all the heaviest locomotives considered and tabulated by the Conference Committees, there is no such engine.

In the Committee's list of heaviest existing locomotives, twelve of the engines have 10 drivers in a group (as in the M-diagram) and only one engine has 12 drivers in a group (as in the A-64 diagram). The Decapods or other engines with 10-driver groups, as assumed in the M-loading, are the prevailing types. The "Duodecapods", as assumed in the A-64 diagram, are practically non-existent.

Any one who has tried to construct a representative modern loading diagram must realize that no single existing locomotive will adequately serve the purpose. The object is to represent the composite stress-producing effects of the various types of governing heavy engines, and this can be accomplished only by a loading diagram that will be a composite picture or symbol of the

* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 638.

stress-producing features of the governing heavy engines. The simplest diagram that will accomplish this should be adopted.

The M-diagram (Fig. 11), while being much simpler than the proposed A-64 diagram (Fig. 2), is also in many respects a better pictorial symbol of the stress-producing features of modern locomotives.

The five-axle group of the M-diagram is a common feature of modern heavy locomotives (Decapods, Santa Fé engines, and Mallets). The six-axle group of the A-64 diagram is an unfamiliar anomaly.

The M-diagram depicts a 2-10-10 locomotive, or a 2-10-0 engine followed by a tender. There are precedents for either arrangement. Compare the following in the list of heaviest locomotives:

Virginian	2-10-10-2
Erie	2-8-10-2
Erie	2-10-2
Pennsylvania	2-10-0

A large number of the governing modern engines have axle loads of approximately 75 000 lb. each (ranging from 72 000 to 86 400 lb.). The A-64 diagram has maximum axle loads of only 64 000 lb.; the M-60 diagram properly has a group of axle loads of 75 000 lb. each.

The A-64 diagram has tender axle loads of only 48 000 lb. each. This is inadequate, both pictorially and mathematically, to represent governing modern engines. Compare the following weights of actual tender axle loads:

	Pounds.
Pennsylvania, I-1-S, Type E	68 000
Chicago, Burlington and Quincy, 2-10-4	64 300
Pittsburgh and Lake Erie, 2-10-4	62 700

Also, compare the following weights of actual axle loads in the lighter groups of drivers in Mallet engines:

	Pounds.
Virginian, 2-10-10-2	62 000
Norfolk and Western, 2-8-8-2	62 800
Union Pacific, 2-8-8-0	59 000
Chesapeake and Ohio, 2-8-8-0	62 600
Erie, 2-8-10-2	64 000

These comparisons show that secondary axle loads of 60 000 lb. each, as used in the M-60 diagram, are more adequate and pictorially representative than secondary axle loads of 48 000 lb. each, as proposed in the A-64 diagram.

The inadequacy of the A-64 diagram and the correctness of the M-60 diagram in the foregoing respect are accentuated by the fact that there is a pronounced trend toward the use of heavier tenders. The M-60 diagram has one group of axles of 75 000 lb. each and another group of 60 000 lb. each. The A-64 diagram has one group of 64 000 lb. each and another of 48 000 lb. each. The former is more adequate and pictorially more correct.

The order of the two groups of axle loads is of minor importance. The M-diagram has the lighter group preceding the heavier group; so has the Erie 2-8-10-2 locomotive; and so will other locomotives if present trends continue.

The A-64 diagram is unduly long and cumbersome in order to make up for the inadequacy of the axle concentrations. The use of proper and adequate

axle loads in the M-60 diagram (Fig. 11) has enabled its length to be reduced to a minimum.

Electric locomotives, which are increasing in number and in extent of use, are better represented by the M-60 loading, both pictorially and in stress consistency, than by any other loading thus far developed.

As a pictorial representation of actual locomotives, the M-diagram is at least as satisfactory as the A-64 diagram. As a recognizable symbolic representation of the stress-producing features of governing locomotives, the M-diagram is in many respects superior to the A-64 diagram.

6.—The Proposed A-64 Simplified Loading Is Lacking in Desired Simplicity and Convenience.—The proposed A-64 simplified loading for spans of more than 40 ft. involves the superimposed combination of a uniform load and a concentration. This means that two stress computations would be required in computing a stress, instead of one. An ideal simplified loading would avoid this inconvenience. The simplified M-loading offered by the writer involves only a uniform load.

The proposed simplified A-64 loading for spans of less than 40 ft. consists of the odd and unprecedented loading of a series of seven driver-axle concentrations, whereas the proposed A-64 locomotive diagram (purporting to represent a "Decapod") has a series of six driver-axle concentrations. In the M-60 loading, the locomotive diagram properly has five axle concentrations in a series, and the corresponding simplified loading for short spans would consist of the identical series of five axle concentrations.

The proposed simplified A-64 loading for shears in spans of more than 40 ft. is awkward for load ratings other than A-64. The loading consists of a uniform load plus a variable excess concentration. The expression for the excess concentration ($E = 128 + 0.5\,L$*) looks simple for the A-64 rating, but loses its simplicity for other ratings. Thus, for A-10, $E = 20 + 0.078125\,L$; and for A-70, $E = 140 + 0.546875\,L$.

Inasmuch as A-64 is a temporary arbitrary rating, apparent simplicity at that loading is not a governing consideration. It is more important that a simplified loading formula be of convenient form for more general ratings.

7.—The Proposed A-64 Simplified Loading for Shears is Unscientific and Yields Incorrect Results for Certain Stresses.—The proposed specification (Article 203*) for the A-64 simplified loading for shears makes the error of defining L as the length of span, whereas it should, for correctness, be defined as the loaded length, or as the length of the loaded segment of the span. This error does not affect the resulting values for end shears but does upset the results obtained for intermediate shears.

It is easily proved that (for any locomotive loading) the equivalent uniform load for intermediate shear at the head of a segment (L) of any span should be practically identical with the equivalent uniform load for end shear in a full span of length (L) equal to the segment (L). By this controlling criterion, the proposed A-64 simplified loading yields incorrect results for intermediate shears, as follows.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2651.

For shear at the mid-point of a 100-ft. span, the A-64 simplified loading, as presented, yields an equivalent uniform load of 13 520 (using $L = 100$, as specified). The proper value would be the same as the equivalent uniform load for end shear in a 50-ft. span, or 12 520 (obtained by using $L = 50$). Accordingly, the former value is 8% too high. Similarly, for shear at the mid-point of a 600-ft. span, the proposed specification yields an equivalent uniform load of 9 250 (using $L = 600$). The proper value (obtained by using $L = 300$) is 8 250. The difference, or error, is 12 per cent.

For counter shear at the quarter-point of a 200-ft. span, the proposed specification yields an equivalent uniform load of 15 520 (using $L = 200$). The proper value (using $L = 50$) is 12 520. The error is 24 per cent. Similarly, for counter shear at the quarter point of a 600-ft. span, the proposed specification yields an equivalent uniform load of 12 110 (using $L = 600$, as defined). The proper value (using $L = 150$) is 9 110. The error is 33 per cent. For smaller ratios of segment length to span length, greater percentage errors (even exceeding 100%) can be shown in the proposed specification.

In structures other than simple spans, requiring segmental loading for maximum stresses, the proposed specification for A-64 simplified loadings will give rise to various ambiguities, particularly in reference to the definition of L. A scientific specification for simplified loadings must define L as the loaded segment, not as the length of span.

8.—The Proposed A-64 Loading and Its Proposed Simplified Equivalents Do not Agree Closely in Stress-Producing Effects.—Some comparisons between the A-64 wheel diagram and the A-64 simplified loadings are as given in Table 14.

TABLE 14.—COMPARISON BETWEEN STRESSES PRODUCED BY A-64 WHEEL DIAGRAM AND A-64 SIMPLIFIED LOADINGS.

Span.	Kind of uniform load.	Wheel diagram.	Simplified loading.	Difference, percentage.
40	U_M	12 160	12 800	+ 5
255	U_M	7 840	7 400	−6
600	U_S	7 420	7 810	+ 5

In Table 14 (and throughout this discussion), U_M denotes equivalent uniform load for moments and U_S denotes equivalent uniform load for shears.

Graphs Showing Inconsistencies of the A-64 Loadings.—In order to facilitate visual appraisal of the A-64 loadings, their equivalent uniform loads are plotted, together with those of the composite loading, in Fig. 12. Fig. 12(*a*) shows the comparisons for center moments and Fig. 12(*b*) those for end shears. The composite loading represents the envelope of the stress-producing effects of the heaviest existing locomotives.

The graphs are plotted on logarithmic cross-section paper to improve the clarity, scale, and balance of the comparisons. A logarithmic scale for the spans gives greater weight to the shorter spans, which are more frequently

used. A logarithmic scale for the comparative loads reduces differences at large ordinates and increases differences at small ordinates so as to be better indicative of percentage differences rather than absolute differences.

The graphs of Fig. 12 show the general unsatisfactory character of the proposed A-64 loadings, namely, their serious inadequacy for short spans; their inadequacy and irregularity for spans of approximately 100 ft.; their extravagance for long spans; and the wide disparities between the A-64 loading and its proposed simplified loading.

Advantages of the M-60 Loading.—Following this outline of the outstanding disadvantages of the proposed A-64 loadings, the following advantages of the M-60 loading may be listed:

1.—The M-60 loading represents more closely the stress-producing effects of the composite of the heaviest existing locomotives (for all stresses in all spans) than any other loading specification thus far proposed;

2.—The M-60 loading is the only loading thus far presented that adequately protects short spans for modern locomotives without unduly penalizing long spans;

3.—The M-60 loading will give a better balanced design for all parts and members of any span (including the floor system) than any other loading thus far proposed;

4.—The M-60 loading is adequate and consistent for spans of approximately 100 ft., where other proposed loadings show inadequacy or irregular depressions in stress-producing effects;

5.—The M-60 loading diagram is a better pictorial representation of the stress-producing features of modern heavy locomotives than the proposed A-64 loading diagram;

6.—The M-60 loading diagram is superior in simplicity, brevity, and convenience for computation, to any other locomotive loading diagram thus far proposed;

7.—With the M-60 loading, alternative simplified loadings can be specified that will be simpler and more consistent than those proposed with the A-64 loading;

8.—The M-60 loading has become familiar to the profession since its adoption in 1923 by the Society's Special Committee on Specifications for Bridge Design and Construction;* and,

9.—The M-60 loading is already widely available, with diagrams and tables to expedite its application, in a considerable number of standard textbooks and reference books for engineers and students.

The proposed A-64 loading, with all its indicated disadvantages, should certainly not be adopted if a better and more convenient loading specification, free from those disadvantages, is available. Unless a loading better than M-60 can be devised, M-60 is the logical loading to adopt for any new specifications.

Simplified Loadings for M-60.—If the form of simplified loadings proposed with A-64 is considered desirable, simplified loadings of similar form can be used with M-60, with some obvious improvements in simplicity, convenience, and correctness. Such simplified loadings for M-60 would be as follows.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 606.

For spans of less than 40 ft., instead of specifying (as is proposed for A-64) a series of seven axle loads of 64 000 lb. each, spaced 5 ft. between centers, a series of five axle loads of 75 000 lb. each, spaced 5 ft. between centers, can be specified.

For spans of 40 ft., or more, instead of specifying (as proposed for A-64) a uniform load of 6 400 lb. per ft. of track plus a concentration of 128 000 lb. (+ $\frac{1}{2}$ L, for shears) a uniform load of 6 000 lb. per ft. of track plus a concentration of 150 000 lb. (+ $\frac{1}{2}$ L, for shears) can be specified.

The results obtained by the use of such simplified loadings would approximate those from the M-60 loading diagram. They would certainly be as simple, if not simpler, than the corresponding loadings specified for A-64.

The writer, however, does not recommend simplified loadings of the form described herein. They possess the disadvantage of requiring a double stress computation, one for the uniform load and another for the concentrated load. Moreover, as previously explained, they are incorrect in form, yielding values too low at or near $L = 100$ in order to avoid objectionable excess at the two extremes of the span range, representing short spans and long spans, respectively.

The simplified loadings just described as possible for use with M-60 can be expressed in the equivalent form:

$$U_M = 6\,000 + \frac{300\,000}{L} \qquad (14)$$

and,

$$U_S = 7\,000 + \frac{300\,000}{L} \qquad (15)$$

Equations (14) and (15) may be compared with the corresponding expressions (Equations (12) and (13)) for A-64 simplified loadings.

In order to yield correct correspondence over the entire span range, the formulas (for either A-64 or M-60) should contain $\sqrt{L}$ instead of L. A deficiency of 8% in resulting stresses at $L = 100$ (or an excess of 8% in resulting stresses in long spans) is too great a price to pay to save the trivial effort of taking the square root of L on the slide-rule (or of using one slide-rule scale instead of another).

Accordingly, the writer proposes and recommends the following improved simplified loadings to be specified with the M-loading:

For the basic M-10 loading:

$$U_M = 600 + \frac{10\,000}{\sqrt{L}} \qquad (16)$$

and,

$$U_S = 750 + \frac{10\,000}{\sqrt{L}} \qquad (17)$$

in which, U_M is the uniform load for moments and U_S, the uniform load for shears. As previously explained, L should be defined as the loaded length, not

the length of span as incorrectly proposed in the A-64 specification (Article 203).

For the proposed standard M-60 loading, the foregoing simplified loadings become:

$$U_M = 3\,600 + \frac{60\,000}{\sqrt{L}} \tag{18}$$

and,

$$U_S = 4\,500 + \frac{60\,000}{\sqrt{L}} \tag{19}$$

It is doubtful whether any simpler alternative loadings, of adequate correctness, can be devised.

The two formulas, Equations (18) and (19), are easily remembered, easily computed, easily tabulated, and easily converted to higher or lower M-ratings. It may be noted that for any value of L, the uniform load for shears (U_S) is always just 900 lb. per ft. greater than the uniform load for moments (U_M). The variable term $\left(\frac{60\,000}{\sqrt{L}}\right)$ is quickly obtained with a single slide-rule operation.

The uniform loads (U_M, and $U_S = U_M + 900$) are easily tabulated for all values of L, or they may be quickly found by slide-rule. Any stress is obtained by applying the uniform load (U_M or U_S) without any additional superimposed excess concentration. Since the A-64 simplified loading requires the additional stress computation for such superimposed concentration, the proposed M-60 simplified loading is an improvement. It requires only one stress computation, instead of two.

The M-60 simplified loading, consisting of U_M and U_S as defined by Equations (18) and (19), will give satisfactory results for all span lengths, without the necessity for excluding spans less than 40 ft.

Nevertheless, in order to secure a more consistently close correspondence between the M-60 loading diagram and the M-60 simplified loading throughout the entire span range, it may be considered preferable to follow the precedent of the proposed A-64 specification by substituting another simplified loading for spans of less than 40 ft. This would consist of five axle loads of 75 000 lb. each, spaced 5 ft. between centers, which is somewhat simpler and considerably more adequate than the corresponding loading proposed for A-64.

Accordingly, the writer offers and recommends the following live load specification based on M-60 to replace the one based on A-64 as proposed by the Conference Committees:

Recommended Specification.—

203.—Live Load.—The minimum live load for each track shall be as shown on Fig. 11, which is hereby designated as the M-60 loading.

If specified by the Engineer, there may be substituted for the M-60 loading the following simplified loadings:

For spans of less than 40 ft., a series of five axle loads of 75 000 lb. each, spaced 5 ft. between centers.

For spans of 40 ft. or more, the following uniform loads (in pounds per foot of track):

$$\text{For moments: } U_M = 3\,600 + \frac{60\,000}{\sqrt{L}}$$

$$\text{For shears: } U_S = 4\,500 + \frac{60\,000}{\sqrt{L}}$$

in which, L = the loaded length, in feet (= span length, for moments; span length, for end shears; and length of loaded segment, for intermediate shears).

The results obtained by the use of these simplified loadings will approximate those from the M-60 loading.

In special locations, where conditions limit the loading to light engines, a lighter loading, as specified by the Engineer, may be used, but in no case lighter than M-45 loading.

Graphic Comparison of M-60 Loadings with Composite.—Fig. 13 records the comparisons (Fig. 13(*a*), for moments, and Fig. 13(*b*), for shears) between the proposed M-60 loading, the proposed M-60 simplified loading (see recommended specification), and the composite of the heaviest existing locomotives.

Fig. 13 shows how the M-60 loadings are free from the outstanding disadvantages of the proposed A-64 loadings. The M-60 loadings are not inadequate for short spans; they are not inadequate for spans of approximately 100 ft.; and they are not excessive for long spans. They are free from the irregular depression or "air-pocket" in the vicinity of $L = 100$.

The correspondence between M-60 and the composite is close throughout the span range, except that M-60 is slightly higher for short spans from $L = 20$ to $L = 60$. This is not due to a defect in the M-loading, but rather to a depression in the composite loading in this range. Judging by past experience, such depression tends to be raised by filling in as new heavy locomotives are added to the list.

Moreover, a slight excess at short spans is an advantage rather than a disadvantage. A deficiency at short spans, as in the A-64 loadings, is a serious defect.

The composite loading includes new heavy locomotives that have been added to the list since the writer computed, in 1922, the composite loading on which the M-60 loading was based. It is significant to note that M-60 fits the new (1930) composite even better than it did the old (1922) composite. The M-60 loading, scientifically devised to smooth out irregularities, is of such type as to anticipate the effects of developments in locomotive design. The writer predicts that the composite of twenty-five years hence will be even more closely represented by M-60 than the composite of to-day.

A comparison of the graphs in Fig. 13 with the corresponding graphs in Fig. 12 shows the superiority of the M-60 loadings over the A-64 loadings. The M-60 graphs show closer agreement with the composite, greater regularity and consistency, avoidance of inadequacy at short spans, avoidance of excess at long spans, and closer correspondence between loading diagram and simplified loading.

Graphic Comparison of A-64 and M-60 Loadings.—Direct graphic comparison of the A-64 and M-60 loadings is presented in Fig. 14 for the loading

diagrams, and in Fig. 15 for the respective simplified loadings. In each diagram, the composite loading is plotted as a standard of reference.

An examination of these comparison graphs clearly shows the marked superiority of the M-60 loadings. The proposed A-64 loadings cannot compare with the M-60 in consistent protection of all span lengths for all stresses without extravagance for long spans. Both the loading diagram and the simplified loading of M-60 fit the composite of the heaviest existing locomotives much more closely, smoothly, safely, and consistently than do the corresponding A-64 loadings.

If ordinary arithmetic scales were used instead of logarithmic cross-section paper for plotting these comparison graphs for moments and shears, the inferiority of the A-64 loadings would become even more obvious.

A closer study of Figs. 12 to 15, inclusive, yields the significant numerical comparisons between the A-64 and M-60 loadings listed in Table 15. These items speak for themselves.

TABLE 15.—Summary Percentage Comparison of A-64 and M-60 Loadings.

Description.	A-64.	M-60.
Maximum inadequacy for center moments	18	4
Maximum inadequacy for end shears	15	2.5
Maximum deviation from composite, for moments	18	8
Maximum deviation from composite, for shears	15	9
Range of variation from composite, for moments	26	12
Range of variation from composite, for shears	21	11
Average variation from composite, for moments	7	2
Average variation from composite, for shears	6	2
Average variation, simplified loading from composite, for moments	7	2
Average variation, simplified loading from composite, for shears	7	1
Range of disparity with respective simplified loading, for moments	11	6
Range of disparity with respective simplified loading, for shears	9	7
Average disparity with respective simplified loading, for moments	2	1.5
Average disparity with respective simplified loading, for shears	2	1
Maximum error of respective simplified loading	More than 100	6

Conclusion.—On the basis of the foregoing studies and comparisons, it should be obvious that the proposed A-64 loadings cannot compare with the corresponding M-60 loadings, in at least ten respects:

1.—Simplicity.	6.—Safe economy.
2.—Convenience.	7.—Regularity.
3.—Scientific correctness.	8.—Consistency.
4.—Representativeness.	9.—Availability.
5.—Adequacy.	10.—Far-sightedness.

There can be no logical justification for adopting the A-64 loading when a far superior loading is already available. Unless a loading better than M-60 can be devised, M-60 is the logical loading to adopt for any new specification.

W. Chase Thomson,* M. Am. Soc. C. E. (by letter).†—In Article 300,‡ "Unit Stresses," why should the permissible unit stress for dead load be greater than that for all other forces acting upon the structure? Many years ago,

* Designer, Dominion Bridge Co., Ltd., Montreal, Que., Canada.

† Received by the Secretary, March 27, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.

Cooper's specification required that live load unit stresses should be one-half those permitted for dead load; but this was to provide an impact allowance of 100 per cent. It was an awkward arrangement, requiring additional figures on stress sheets; and it was faulty in that the different permissible values for main materials were not taken into account in proportioning the details. Impact was later introduced into specifications, with the idea of increasing the live load to an equivalent dead load. This has been considered as a decided step in advance of earlier methods used in proportioning structures. It is to be hoped, therefore, that the proposed adoption of separate unit stresses for dead load will be reconsidered.

In Article 301,* the second-degree formulas for axial compression and for compression flanges of girders are regrettable from the designer's point of view. In the case of axial compression, the curve, as represented by the formula, is only used for ratios of $\frac{L}{r}$ between 40 and 120; and a straight-line formula, such as $17\,000 - 75\,\frac{L}{r}$ (maximum 14 000), gives, between these limits, practically the same results, and with much greater facility.

In Article 408,† the rules for determining the net section of riveted tension members may be sufficiently satisfactory to use when investigating the strength of an existing structure; but they are entirely too complicated and, therefore, unsuitable when designing a new structure. In the application of these rules it has been found that, when the diagonal distance between centers of staggered rivets exceeds the distance between adjacent gauge lines by 30%, the net sections, (*a*) and (*c*), are nearly equal. It is suggested, therefore, that Article 408 be revised to read as follows:

"The net section of riveted tension members shall be determined by deducting as many rivet holes as can be cut by a right section, provided the diagonal distance between centers of staggered rivets shall be at least 40% greater than the distance between adjacent gauge lines; otherwise, allowance shall be made in each component part of the member for as many rivet holes as it contains gauge lines.

"Rivet holes shall be assumed $\frac{1}{8}$ in. larger than the nominal diameter of the rivet".

In Article 419,‡ the formulas for computing shear in compression members are far too complicated for practical use; and it is extremely doubtful whether the results obtained thereby will be more nearly correct than a simple assumption that the shear is equal to 3% of the axial stress.

Article 428,§ specifies that flange plates shall be equal in thickness, or that they shall diminish in thickness from the flange angles outward. The writer can see no reason for these requirements, except that they have usually been included in specifications during the past thirty or forty years. Sometimes, it is desirable and advantageous to reverse the order, thus increasing the thickness from the flange angles outward. This Article also specifies that, when flange plates are used, at least one plate on each flange shall extend the full

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2654.
† *Loc. cit.*, p. 2656.
‡ *Loc. cit.*, p. 2657.
§ *Loc. cit.*, p. 2659.

length of the girder. In most cases, it would seem to be an unwarranted extravagance to make the first plate on the bottom flange of full length.

In view of the fact that plate girders seldom or never fail by the buckling of their web-plates, the formula for spacing of intermediate stiffeners (Article 433*) seems to be unnecessarily severe. It should be sufficient to specify simply that the distance from center to center of intermediate stiffeners shall not exceed 6 ft., or the depth of the web-plate. The phrase, "depth of the web between flanges", is somewhat ambiguous, and may erroneously be taken as the clear distance between the vertical legs of the flange angles.

JONATHAN JONES,† M. AM. SOC. C. E. (by letter).‡—The value of a specification sponsored jointly by the Society and the American Railway Engineering Association would appear to be conditioned as follows: (1) That there be a consensus of opinion among American structural engineers that it expresses the best, or at any rate acceptable and up-to-date, designing practice; and (2) that a substantial number of railroad systems (all, in fact, except a few whose peculiar requirements prevent adherence) will actually adopt and issue it to govern their purchases.

It seems difficult to believe that either criterion will be met by a specification differing in many important particulars from specifications heretofore in use, either (*a*) with respect to methods of statement (as impact, latticing, etc.); or (*b*) with respect to fundamental allowances (as dead load unit stress, silicon steel allowable stress, etc.), unless effectively "sold" to the profession by the Committees. It is unfortunate that the Committees should not, for first publication, have appended to many, if not most, of the clauses a brief résumé of the majority position leading to their submission. The discussions and objections, of which there undoubtedly will be many, could be more brief and more effective if the Committees' reasoning had been revealed.

Doubtless some of the changes in formula or method proposed by the Conference Committees would thus be shown to have resulted from new experiments, analyses, or reconsiderations most properly taken into account. This specification is, however, not a record of research, but a working tool, and should be of the greatest allowable simplicity. Assuming, for example, that one could, by research and analysis, devise an entirely correct formula for latticing, taking into account all the variables that may exist, and that the profession should agree that it was entirely correct; it would still be of value as a standard of reference only, and not as a specification clause. The latter still must be pared down to the simplest possible statement, giving values (for the range intended) not too far different from those which would be yielded by the perfect standard of reference. This point of view, it need hardly be added, is perfectly sympathetic toward research and reconsideration of every nature; but it defends the use of formulas unscientific in make-up, provided they yield acceptable results with a minimum of effort.

To be specific, the writer would regret to see Article 419,§ comprising two somewhat tedious formulas for latticing, adopted instead of the simpler

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2659.

† Chf. Engr., McClintic-Marshall Co., Pittsburgh, Pa.

‡ Received by the Secretary, April 1, 1930.

§ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2657.

Article 68 of the A. R. E. A. Specifications of 1925, until shown wherein the latter has failed to give acceptable results.

The change in basic dead load stress of from 16 000 to 24 000 (Article 300*) involves a lightening of structures, since the proposed impact formula gives, for spans of less than 130 ft., a lower percentage than that of Article 28 of the A. R. E. A. Specifications of 1925; and for spans of more than 130 ft., it never gives an increase greater than 10 per cent. To be sure, the basic wheel load is increased about 7%; but this is, and probably will have to be, subject to change by the purchaser, so that increase of section from that source is not assured. Since dead loads almost always produce secondary stresses, and since it is not an accepted fact that bridges are being built too heavy, neither the reduction of section nor the added complexity of calculation—which applies to drafting room calculations of rivets, pins, lattice-bars, and details quite as much as to sectional area—has any appeal for the writer. Likewise, the use of the "kip" expression instead of "pounds" in tabulating the units.

Article 301(*a*)† and 301(*b*)† might well be re-considered and re-cast. It is quite possible that the American Society for Testing Materials may soon adopt a specification for medium steel (33 000 minimum yield point) as an optional alternative for the present bridge steel. Such a medium steel has been used recently, and should be used increasingly, for important structures; and the matter will probably be cleared up in time to take such an option into account through alternative unit stresses in Article 301. In that case the bridge specifications would recognize specifically four definite steels, *viz.*, bridge steel (present grade), medium steel, silicon steel, and nickel steel. It would be well worth the additional paper and ink to tabulate all the allowable unit stresses for each of the four, and thus remove every trace of ambiguity. That paper and ink could, and, in the writer's opinion, should, be saved by eliminating the steel specifications except by reference to the A. S. T. M. Specifications as is very properly done in Article 1201.‡ Much needless doubt and making of comparison readings, at shop and mill, would be saved thereby.

Furthermore, the writer does not favor the proposal of Article 301(*b*) that high-strength steels now known, and others yet to be introduced, shall be computed on the basis of their specified yield points. There are too many other characteristics affecting the suitability of a hard steel for bridge use to justify such a blanket prescription. To take the specific case of silicon steel, enough is known of it to establish 24 000 as a conservative basic unit stress and, in the writer's opinion, to render unnecessary the refinement of modifying the denominator of the compression formula. Is the proposed modification of the denominator essential within the limits of ordinary design, and if so, cannot a definite formula be used for this and another for nickel steel, instead of adopting a wording which is at least susceptible to misconstruction?

Other steels may be considered from time to time as they are offered, and their specifications defined and accepted on bases that then seem proper; but a unit stress basis should not be written now, nor until after some experience with the particular steel is available as a basis for judgment.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.

† *Loc. cit.*, p. 2654.

‡ *Loc. cit.*, p. 2676.

The prescription of 600 d for rollers (Article 301) requires amplification, because it gives values that are too high for the large single end-bearing rockers, with radii of several feet, which are coming into increasing use instead of roller nests. The work which has been done in this direction at the University of Illinois might well be taken into account in amplifying this clause.

The writer wishes also to offer a number of detail suggestions made by a group of designers who will have to resolve, in their daily work, any uncertainties that may remain in the specification when adopted.

Neither the opening page, "Questions to be Answered,"* nor Article 2† of Section A, seems orderly and complete, and the term, "Proposals," might better be eliminated from a specification for "Design and Manufacture," replacing it in the Section title with "General Plans" and re-writing and amplifying Article 2 along these lines:

"2. General plans shall in all cases be prepared by the Company. They shall show profile or profiles of crossing, elevation, alignment, and spacing of tracks, and super-elevation if any, depth and variation of water, nature of river bottom, survey of tracks or streets crossed, prescribed under- and over-clearances, presence of old structures, traffic conditions, right of way, storage and working areas, and any other facts affecting the design or erection. They shall further show length and type of spans, angle of skew, and type of floor, and the general dimensions of masonry.

"If the design drawings are prepared by the Company, as is recommended, they shall show the dead load, the live load, the general dimensions, stresses, sectional areas, and make-up of all parts, the several grades of material, the general dimensions and nature of bearings on masonry, typical details of splices and connections and of all castings, railings, drainage details, and other appurtenances, and a standard of quality for the shop and field paint.

"If the design drawings are to be prepared by the Contractor or by bidders for the contract, to the general plans furnished by the Company shall be added the live loads to be used, the several grades of material as preferred, or as prescribed, the desired provision for corrosive conditions if such conditions exist, the rivet sizes, any special requirements as to railings, drainage, or other appurtenances, and a standard of quality for the shop and field paint."

Article 3.†—The size of shop drawings should not be specified rigidly, because a larger size is essential on large structures. A size of 29 by 41 in. has been found to be a desirable alternative.

Article 103.‡—What minimum margin of safety is desired against overturning?

Article 105.‡—What width of car is necessary to determine side clearance?

Article 107.‡—Where ambiguity of stresses exists, shall the designer be advised as to when elastic calculations based on the finished sections will and will not be required?

Article 108.‡—End floor-beams should preferably be provided with adequate stiffeners and connections to permit jacking up the entire bridge for repair or replacement of bearings.

Article 203.§—It seems a pity that, if any change is to be made, the work so thoroughly performed by D. B. Steinman, M. Am. Soc. C. E.,‖ and so gener-

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2648.

† *Loc. cit.*, p. 2649.

‡ *Loc. cit.*, p. 2650.

§ *Loc. cit.*, p. 2651.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 606.

ally approved, should here be passed over. Were not the arguments against the adoption of the Steinman loading, principally against any change at all, which the Committees have still to overcome? If a new loading is to be adopted, it should be accompanied by the usual moment and shear tables and diagrams and a table of conversions to E-loading.

In the last paragraph of Article 203, the Conference Committees recommend that:

"For girders and trusses of spans carrying more than one track, live loads shall be assumed as follows:

"For two tracks, full live load.

"For three or more tracks, full live load on the two tracks nearest the girder or truss, and 90% full live load on all other tracks."

Is the saving of 10% of the live load worth while on the distant tracks of such bridges?

*Article 204.**—The specification states:

"For girders and trusses of double-track bridges, the impact shall be taken from the full live load on one track only. The impact shall be only that from the live load on the track nearest the girder or truss".

What, then, is the impact for design of the truss, the entire impact load found by formula, or as much of this load as is transferred by the floor-beam to the nearer truss? The uncertainty approximates 20 per cent. The same ambiguity affects the next paragraph, for bridges of three or more tracks.

*Article 205.**—The table of percentages for centrifugal force (Table 1) is an excellent example of simplification as previously recommended in this discussion. Here is a direct and practical working tool, rather than an accurate but cumbersome formula.

Article 211.†—A more logical wording would first define the alternative stresses, and then advise which is to be used. In cases where the alternatives are fairly equal, but compression governs, should it not be prescribed that net section is to be protected for the tensile alternative? Without a further statement here as to the stress for design of connections, Article 407‡ on that subject is not explicit.

Is the penalty imposed on reversals by this clause really essential, and if held essential as to connections, should it not be removed as to sectional areas? It has the effect of seriously diminishing the economy of cantilever and continuous trusses, in which many important members sustain reversal. As a matter of fact it is frequently not observed and, where there is no ambiguity, is almost certainly not worth what it costs.

Article 213†.—It would be well to be explicit here as to how the allowance, in Article 300, of 50% increase in dead load unit stress, is to be applied to these combinations. With 24 000 lb. per sq. in. allowed for dead load, this 25% increase seems excessive.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.
† *Loc. cit.*, p. 2653.
‡ *Loc. cit.*, p. 2656.

*Article 214.**—The sentence, "other secondary stresses shall be considered", is not complete. If it means that they shall be considered as primary stresses, it results in a radical addition to the time required for calculations and to the resulting areas. If it means something short of that, it is not much of an instruction to the designer.

Article 302.†—This Article is superfluous if Article 300 is retained, as the two yield identical results.

Article 401.†—A minimum thickness of ⅜ in. and of ½ in. for gussets, would be more suitable to almost every case that will arise. A thickness of 5/16 in. might be acceptable for webs of rolled beams and channels. If it is felt that the added resistance of copper-bearing steel should receive recognition at this time, a reference at this point would be appropriate.

Article 407.‡—Nothing has been given in any specification of which the writer is aware, on the design of gusset-plates for riveted trusses. In every such case, certain critical sections must be tested for a combination of stresses, in order to determine that the dimensions and thickness of the plate are correct. The criteria are left wholly to the judgment of the detailer. There is perhaps no single item in which so much of the adequacy and economy of the design are left unprescribed. The writer feels that something should now be offered by the Committee to fill this very noticeable gap.

Article 409.‡—In part, this specification states that "If angles in tension are connected so that bending cannot occur in any direction, the effective area shall be taken as the net area of the angle". This may be so interpreted that a member of two angles, attached to each other by tie-plates or lacing, and connecting each to a gusset, may be considered as two complete angles. This seems, however, subject to dispute unless the statement is made more explicit.

Article 410.‡—The second sentence, prescribing a special form for long rivets, belongs under workmanship rather than design, and is capable of being made more definite.

Article 414.§—A compression splice of 25% should be sufficient for perfect planing, but it does not contain enough margin to be generally applicable until all shops are brought up to the highest standards of workmanship; 50% is advisable.

Article 427.‖—One-eighth of the gross section of the web, if properly spliced, may be considered as net flange section, and one-sixth as gross flange section.

Article 428.¶—If girder flanges are preferably to be made without cover-plates, then it seems excessive to extend the first bottom cover-plate full length. Considering the inaccuracies of computations it would seem that 18 in. beyond the theoretical ends of cover-plates is not wasteful.

Article 429.¶—The requirement that no two members be spliced at the same cross-section makes field splicing cumbersome and eliminates the possibility of facing the spliced ends for bearing.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2653.

† *Loc. cit.*, p. 2655.

‡ *Loc. cit.*, p. 2656.

§ *Loc. cit.*, p. 2657

‖ *Loc. cit.*, p. 2658.

¶ *Loc. cit.*, p. 2659.

*Article 430.**—Shall web splices be designed to develop the full value of the web in shear and bending or to protect the value of the girder? The latter prescription is adequate; it will frequently obviate the use of moment plates where splices are far from the point of maximum bending moment.

Article 439.†—Is the expression "and towers shall be formed" mandatory; in other words, must all pairs of bents be braced together into towers? The fourth paragraph implies otherwise; but it, in turn, is incomplete without a definition of a "short bent".

Article 442.‡—The provision for floor expansion is not sufficient. Disregarding temperature changes, the differences between the fabricated stringer lengths and the shortened bottom chord lengths cannot be carried even 200 ft. without introducing real trouble in erection. When silicon chords are used (and they are being used for spans as short as 200 ft.) the difference is still greater because the amount of camber shortening is greater. Provision for floor expansion should be based upon erection conditions and chord stresses, and occasional double floor-beams should be favorably regarded.

Articles 443-445.‡—The last sentence of Article 445 would be a proper beginning sentence for Article 443. It should be provided that hinge effect may be secured by a pin, a convex slab with line bearing, or otherwise. As to expansion, spans somewhat longer than 70 ft. can properly be arranged to slide on bronze plates and some discretion should be permitted. The California State Division of Highways is conducting a research in this field. These Articles are incomplete in making no reference to large end rockers, which in many instances are preferable to roller nests. Probably 1½ in. would be a better minimum than 1 in. for cast-steel pedestals. The three Articles together cover a subject which represents a surprisingly large proportion of the total cost of any steel bridge, and they should be very carefully re-drafted to represent, in a better way, the present status of the subject.

In addition to the foregoing, the sections respecting "Workmanship,"‡ and "Weighing and Shipping"§ should be thoroughly criticized before adoption. The writer feels that the Society will overlook an opportunity if it continues to prepare specifications for a manufacturing process, through a committee on which the manufacturer is not represented. The best interests of the purchaser and the actual present-day state of the art, are apt not to feature a specification thus drafted.

P. G. LANG, JR.,‖ M. AM. SOC. C. E. (by letter).¶—It appears to the writer that, in the preparation of a new and separate specification for steel railway bridges, the Conference Committees have acted in a manner at variance with the purpose for which they were appointed. The apparent intention was that the labors of these Committees would be in the direction of harmonizing specifications already in existence, with a view to the ultimate development and adoption of one standard specification. The actual result of their work is

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2659.
† *Loc. cit.*, p. 2660.
‡ *Loc. cit.*, p. 2661.
§ *Loc. cit.*, p. 2666.
‖ Engr. of Bridges, B. & O. R. R., Baltimore, Md.
¶ Received by the Secretary, April 7, 1930.

an entirely new specification, distinct and different from specifications previously in use or under consideration. The situation, therefore, has been complicated rather than simplified.

The number of firms engaged in the fabrication of structural steel railroad bridges and other structures of a similar character is relatively small. Their facilities are designed with a view to mass production. It is obvious that, under such conditions, agencies which tend to simplify practice and increase uniformity constitute a source of economy in time and money to producer and user. Under these circumstances, it seems evident that the efforts of the Engineering Profession should primarily be directed toward harmonizing existing differences rather than in the creation of a new series of requirements.

The proposed specification predicates no limitation as to the span length of bridges to which it is applicable, but is apparently designed for universal use in connection with metallic structures.* Such procedure appears inadvisable from any standpoint, and is not justified by American conditions. Recent investigation of railroad bridges in that part of the United States north of the Potomac and Ohio Rivers and east of the Mississippi River indicates that, of permanent bridges (that is, all classes except timber trestles), 57% of present spans are less than 31 ft. long, and 40% range from 31 to 120 ft.; that is, 97% of permanent railroad bridge spans in this region are less than 120 ft. long. Spans between 120 and 250 ft. in length comprise less than 3% of the total, and the number exceeding 300 ft. bears an infinitesimal ratio to the total. Spans which attain or exceed this length are so few, and the conditions surrounding such construction are usually so unique, as to justify the development of a new specification in each case.

The proposed specification contemplates the introduction, for design purposes, of an entirely new live load (Article 203†), and, in so doing, revives a question which has long been under discussion. While the Cooper series has held the preference of railroad bridge designers for many years, its use in this field has been challenged repeatedly, and numerous alternate loadings have been suggested. In each case, investigation has demonstrated the superior merits of the Cooper series. In no case has the proposed alternate loading been found to represent actual railroad loadings of all types more accurately than the Cooper series, and this observation applies with equal force to the loading contemplated in the proposed specification.

As a result of the long and wide acceptance of the Cooper loadings as a measure for rating railroad bridges, the majority of such records have been compiled on this basis, and the significance of this terminology is generally understood, in at least some degree, by lay members of railroad organizations. Consequently, any deviation from the Cooper loadings will inevitably be productive of great expense and confusion, and the adoption of any loading as a substitute should not be considered unless motivated by reasons of an extremely cogent nature. Such reasons are not apparent in connection with the loading proposed in Article 203.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2641.

† *Loc. cit.*, p. 2651.

It is observed that Article 204* specifies the use of the following impact formula:

$$I = S \frac{400 - \frac{L}{2}}{400 + L}$$

During 1929 the writer had occasion to review the general subject of impact in railway bridges, and the various formulas which have been adopted to represent the factor in question. This research included a historical review of developments along these lines during the past half century. Within that period this topic has been investigated exhaustively, from an empirical standpoint in the accumulated results of actual experimentation, and in attempts to analyze mathematically the dynamic increment in its relation to the other forces to be considered in structural design. As a result of this investigation, a variety of formulas designed to represent dynamic stress have appeared, and, in some cases, have been submitted to the practical test of use in bridge design and construction.

Of these probably the most widely used is the formula:

$$I = S \frac{30\,000}{30\,000 + L^2} \qquad (20)$$

This equation is now embodied in the current edition of the American Railway Engineering Association General Specifications for Steel Railway Bridges.

A canvass undertaken in 1923 indicates that, of 92 roads reporting, 79 used the American Railway Engineering Association Bridge Specifications. Consequently, it is evident that the impact formula contained in this specification is used extensively at present in the design of railroad bridges. This formula has long been used, with entirely satisfactory results.

While it is not the intention to deprecate further investigation into this matter, and the possible adoption of other formulas as the results of future research and experience may justify, actual experience and comparison of various formulas indicate that the ultimate effect upon the completed structure is very small. This is especially the case with respect to short spans, and, as previously stated, investigation indicates that more than one-half the existing railroad bridge spans are less than 31 ft. long, and 97% less than 120 ft.

In view of the relatively minor differences which the use of various impact formulas represents in the completed structures, it would appear that this phase of bridge design should lend itself with peculiar readiness to standardization and uniformity, and that, before proceeding to recommend an entirely new formula, the Conference Committees should consider the possibilities of extending the adoption of formulas which are already in wide and successful use.

It is noted that the properties of structural steel, as set forth in Article 809,† include a yield point of 30 000 lb. per sq. in., and an ultimate strength of 55 000 to 65 000 lb. per sq. in. While these figures are in general conformity with requirements embodied in specifications prepared and published several years ago, their appearance in a new specification, designed to represent the most modern practice, appears somewhat of an anachronism.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.
† *Loc. cit.*, p. 2668.

A review of test reports covering more than 10 000 tons of steel work for bridges on the Baltimore and Ohio Railroad indicated that less than 1% of the material actually had a yield point less than 35 000 lb. per sq. in. In view of the foregoing condition, that company now specifies material to meet these requirements; that is, elastic limit, 35 000 lb. per sq. in. minimum, and ultimate strength, 55 000 to 70 000 lb. per sq. in.

The production of structural metal of minimized corrosive susceptibility has been a subject of considerable study and research. Based on service experience, it is unquestionable that a small copper ingredient greatly increases the resistive quality of structural metal to corrosion. It is accordingly the writer's definite view that a modern bridge specification should provide for approximately 0.2% copper content, especially as the increased expense due to the addition of copper is small in proportion to the cost of the steel work and the known benefit incident to the use of copper.

The writer's definite conclusions are as follows:

Article 102.—Types of Bridges.*—Pin-connected trusses should not be used except in unusual cases.

Article 203.—Live Load.—The proposed live load offers no advantage over the Cooper loadings; further, the simplified or alternate loadings do not approximate the A-64 loading as closely as the A-64 loading approximates the Cooper loadings.

Article 204.—Impact.—The proposed impact formula, when combined with dead load, live load, and other stresses, will not produce a bridge structure materially different from that obtained when the Cooper loadings, with the impact formula, $I = \frac{30\,000}{30\,000 + L^2}$, plus dead load and other stresses, are used.

Article 300.†—Unit Stresses.—Provision for increasing unit stresses for dead load should be omitted, and, instead, a paragraph reading about as follows should be substituted: "When designing a structure, consideration should be given to its ultimate carrying capacity." The same comment applies to provisions for overload (Article 302‡).

Article 301(b).§—High Tension Steel.—It is not clear what is meant by "structural grade" steel. Presumably this is primarily designated by the minimum yield point. If the yield point varies from that specified, apparently the stresses should be changed to correspond with the yield point, whether or not the steel is so-called "high tension."

Conclusion.—On the whole it is not felt that this specification is to be preferred to the American Railway Engineering Association General Bridge Specifications for Steel Railway Bridges, Third Edition, August, 1925. It does not represent a definite step in advance, but simply offers something differing in minor detail.

Therefore, since need for this new specification is not proved in itself, all interested engineers should frankly admit that the existing A. R. E. A. specification is satisfactory and should continue to use it.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2650.

† *Loc. cit.*, p. 2653.

‡ *Loc. cit.*, p. 2655.

§ *Loc. cit.*, p. 2654.

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PAPERS AND DISCUSSIONS

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EXPERIMENTAL RESEARCH ON VIBRATION DAMPERS AND INSULATORS

Discussion*

BY MESSRS. W. M. DAWLEY AND STEPHEN E. SLOCUM.†

W. M. DAWLEY,‡ M. AM. SOC. C. E. (by letter).§—A consideration of some of the fundamental equations of motion as derived by Mr. J. W. S. Rayleigh‖ leads one to question whether the arrangement of the test apparatus adopted by Mr. Slocum was the best for the purpose (see Fig. 1).¶ The test pad at the center and the unbalanced rotor near the quarter-point of the transmitter I-beam with the plane of rotation at right angles to the length of the beams, would set up a harmonic overtone vibration in the beams having a period one octave above the normal period of the undamped beam, the latter period being more or less damped out. The test pad, therefore, rests at the nodal point in so far as the horizontal vibrations are concerned and does not respond fully to the impressed vibrations.

The test load applied to the one-third point of the H-beam acts as a damper to produce a node at that point. On the assumption that the flexibility of the test pad permits horizontal vibrations of that end of the H-beam resting on it, the period would be one octave above the normal period or "gravest node" of vibration of the beam with both ends fixed, but otherwise free.

In discussing this phase of the problem, Messrs. J. Ormondroyd and J. P. Den Hartog, have shown that a simple steel bar, 2 in. by ¾ in. by 60 in., lying flat and simply supported at the ends and supporting a 26-lb. direct-current motor at its center with two 8-lb. weights, one attached at the quarter-point, and the other at the three-fourths point, of the length, had natural frequencies of 750 and 2 800 rev. per min.**

* Discussion of the paper by Stephen E. Slocum, M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Author's closure.

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§ Received by the Secretary, February 11, 1930.

‖ "Theory of Sound," by J. W. S. Rayleigh, Second Edition, 1894, reprinted 1926.

¶ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2111.

** *Transactions*, Am. Soc. Mech. Engrs.; Applied Mechanics, Vol. 50 Paper 7; abstract in *Mechanical Engineering*, December, 1928, p. 975.

It would seem to have simplified the interpretation of results if the rotor, test pad, and load from the testing machine had all been placed in the same vertical plane. This plane would pass through the test pad at right angles to the beams, with the center of the rotor on a level with the steel plate under the test pad for horizontal motion tests, and vertically under the test pad for vertical motion tests.

The use of such an unbalanced rotor as that described by the author introduces undesirable variations which confuse the analysis. Up to a certain critical speed the motor armature, with its unbalanced pulley, will rotate about its geometric center and the force that produces vibration will be only that of the unbalanced component, a few ounces or pounds. At speeds greater than this critical speed the vibration will involve the whole mass of the armature and pulley as well. The tendency then is for the entire rotating part to revolve about the common center of gravity of armature and unbalanced weight, which is not the geometric center of the shaft. Since the unbalanced pulley is at one end of the motor shaft, that end of the motor will vibrate more than the opposite end at high speeds, thus tending to produce a rotary motion similar to that of a rotating pendulum, except that the axis of rotation is horizontal instead of vertical.

There is also the further complication that the phase of the vibration at speeds greater than the critical speed is almost, but not exactly, opposite the phase at speeds less than the critical and that the amplitude of vibration changes at the critical speed, being less at the higher speeds, as shown in Fig. 14.* The effect of one or several unbalanced weights rotating in different parallel planes has been described at considerable length by Mr. R. Eksergian† in connection with the problem of dynamically balancing the reciprocating and rotating parts of a locomotive.

Viewed from another angle, consider the H-beam and the testing-machine load as a steady mass which it is desired to set in motion by means of the unbalanced weight on the motor pulley, and that the amplitude of the receiver graphs is a fair measure of the energy that has survived the trip through the transmitter I-beams and test pad.

There is not much variation in the energy transmitted under the conditions illustrated by Figs. 2,‡ 7,§ 9,‖ 10,‖ 11(*b*),¶ and 11(*d*).¶ That is, approximately the same percentage of the energy developed by the unbalanced rotor is transmitted notwithstanding the fact that the more or less flexible test pads permitted the transmitter I-beams and plate to vibrate more or less freely. In other words, the amplitude of the vibrations set up in the transmitter beams is not the proper function to compare with the amplitude of receiver graphs to determine the efficiency percentage of the insulator pads.

The flexible I-beams interposed between the rotor and the steel plate on which the test pad rests should be eliminated, or the energy lost in setting the

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2127.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 51, No. 17; Railroads, Vol. 51, No. 5.

‡ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2114.

§ *Loc. cit.*, p. 2118.

‖ *Loc. cit.*, p. 2120.

¶ *Loc. cit.*, p. 2123.

transmitter beams in vibratory motion should be ascertained by comparing the amplitude of the unloaded transmitter with that of the transmitter with the test pad and testing machine load applied.

However, transmitter graphs for $f = 1\,100$ shown in Fig. 7(*a*) and Fig. 7(*b*), seem to be inconsistent with this latter point of view, because the damped vibration of the lightly loaded transmitter shows a greater amplitude than the unloaded transmitter graph for the same speed shown in Fig. 14.

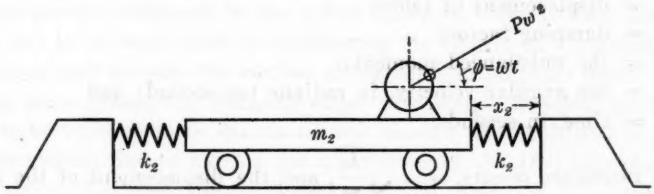

FIG. 18.

In the vibration research laboratory at Leland Stanford, Jr., University the shaking table* consists of a heavy platform mounted on four street-car wheels, and anchored by springs (in compression) to heavy concrete piers (see Fig. 18). The platform is made of 8-in. H-beams bolted and welded together; its dimensions are 10 by 12 ft. with a weight, m_2, of approximately 6 000 lb. In order to reduce the friction of the table motion to a minimum, the street-car wheels have been ground concentrically with their axles and mounted on ball bearings.

In considering the type of motion to be imparted to this table it was decided that it must be "a civilized" disturbance, a motion of the table that can be repeated again and again with great accuracy; and it must, if possible, be a motion that can be treated mathematically with some degree of comfort, in order that the accelerations and velocities involved may be known quite accurately.

These considerations have led to the adoption of two distinct types of linear motions, namely, a damped, free vibration of the table, resulting from an elastic impact of a pendulum; and a continuous, forced vibration, set up by a revolving unbalanced fly-wheel mounted on the shaking table. The two types of resultant motion are, of course, quite different. In the case of the pendulum, maximum acceleration occurs when the initial displacement is quite small and the succeeding maxima of the accelerations are each less than the preceding, the difference between the first and second being considerably greater than the difference between the succeeding maxima. The maxima of the accelerations are at maximum amplitudes. The damping of the free vibration is a combination of constant damping and velocity damping.

In the case of the unbalanced pulley the resultant vibration of the table is very nearly a simple harmonic motion or sine curve.

The equation of motion is:

$$m_2\, x_2'' + c_2\, x_2' + k_2\, x_2 = P\,\omega^2 \cos \omega\, t \ldots\ldots\ldots\ldots\ldots (4)$$

* See Lydik S. Jacobsen, in *Bulletin*, Seismological Soc. of America, Vol. 19, No. 1, March, 1929, Fig. 18.

Integrated, this equation becomes:

$$x_2 = \frac{P\,\omega^2 \cos(\omega\, t - \psi)}{[c_2^{\,2}\,\omega^2 + (k_2 - m_2\,\omega^2)^2]^{\frac{1}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which,

ψ = arc tan $\dfrac{c_2\,\omega}{k_2 - m_2\,\omega^2}$;
m_2 = mass of table;
k_2 = characteristic of anchor springs;
x_2 = displacement of table;
c_2 = damping factor;
P = the unbalanced moment;
ω = the angular velocity, in radians per second; and
t = time, in seconds.

When resonance occurs, $\omega = \sqrt{\dfrac{k_2}{m_2}}$, and the displacement of the table is,

$$x_2 = \frac{-\,P\,\omega \sin \omega\, t}{c_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The pulley was mounted on the table independently of the motor, and, judging from the photographic illustration, it appears to be 2 ft. or more in diameter. It is loaded with eight pigs of lead braced under the rim and a shot tank located between the spokes which can be filled with any desired amount of shot and emptied without stopping the rotation, thus varying the amplitude at will by altering the amount of unbalance without changing the speed. The motor is also set on the table and is belt-connected with the pulley.

Fig. 19* illustrates quite clearly how the maximum amplitude of vibration of the shaking table, which simulates Mr. Slocum's unloaded transmitter, varies with the speed of rotation of the unbalanced pulley.

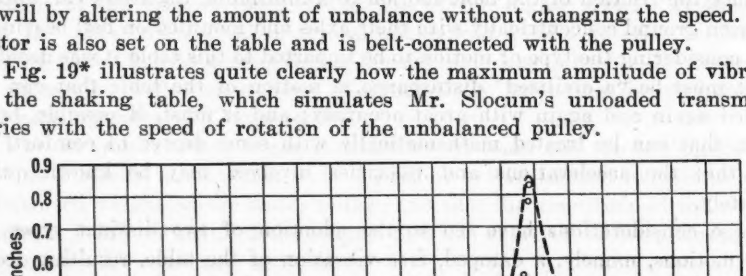

FIG. 19.—THEORETICAL CURVE FOR VISCOUS DAMPING, PROPORTIONAL TO x'_2 ACCORDING TO L. S. JACOBSEN.

It would seem then that the best measure of the value of a "vibration damper or insulator" would be its capacity to absorb or dissipate the kinetic energy of the applied forced vibrations.

* See Lydik S. Jacobsen in *Bulletin*, Seismological Soc. of America, Vol. 19, No. 1, March, 1929, Fig. 16.

To measure this capacity a wave of known kinetic energy should be applied to one side of the material under test and the surviving energy of the same wave emerging from the opposite side should be measured by suitable means. For simplicity in analysis the form of the impressed wave should be simple harmonic motion, although it may be possible to analyze the resultant record of two concurrent harmonic motions of different periods, amplitudes, and epochs, provided the period, amplitude, and phase of either one is known or is capable of independent determination.

Experiments conducted by the writer in June, 1925, at East Binghamton, N. Y., may be of some value in considering the subject of vibrations set up in and transmitted through the surface of the earth at no great depth, since they show to some extent the damping effect of distance traveled, on amplitude. These tests were made to determine the increase in amplitude of vibrations in a greenhouse, due to moving the main track of the Delaware, Lackawanna and Western Railroad from a location 122 ft. away to a parallel location 80 ft. away.

The U. S. Topographical Survey map for this region shows the location to be at the narrowest part of the Susquehanna River Valley, the hills rising on either side of the narrow river bottom to heights of from 1 500 to 1 600 ft. The elevation of the greenhouse floor and adjoining field is 840 ft., and the normal surface of water in the river, 831 ft., above mean sea level.

This topographical layout would appear to preclude the possibility of there being any soft or yielding material underlying the sandy loam soil, mixed with the rather large pebbles on which the railroad embankment and greenhouse are situated. The probability is that it is gravelly loam, then sand, and then gravel down to hardpan, clay and boulders mixed, and, finally, bed-rock.

The instrument used in making the tests consists of a pendulum of 93.5 lb. (42.2 kg.) weight suspended by piano wire from an engineer's transit tripod. It has an undamped period of 2.222 sec., or 27 complete oscillations per min. For portability the weight is made of four accurately turned cast-iron disks, each $8\frac{3}{4}$ in. in diameter and $1\frac{3}{4}$ in. thick, mounted in two pairs on a 1-in. steel bolt which passes through an accurately bored hole in the center of each. The bolt has a taper of a few thousandths of an inch, so that each disk fits tightly. The upper pair of disks is separated $\frac{1}{2}$ in. from the lower pair by three brass studs to permit two thrust arms, at right angles to each other, to be attached to the center of the steel bolt at the center of percussion of the pendulum. The entire arrangement is securely clamped together on the bolt by means of a nut countersunk in the top disk. By means of these thrust arms and a system of multiplying levers attached to the steel bed-plate which is in contact with the earth, the relative motion of the earth and pendulum is magnified 148.6 times.

Record was made on smoked paper passing over a brass cylindrical drum by a fine steel needle-point in the extremity of each of the recording levers. One of the levers is bent at right angles, in order to record the two rectangular co-ordinates of motion on the same smoked sheet at a distance of 3 in. apart. To reduce the friction to a minimum all connections between thrust

arms, multiplying levers, and recording levers are made either by springs or by polished conical steel pivots in V-shaped white sapphire jewels.

All moving parts are of duraluminum or aluminum. The weight of the moving parts of the amplifying lever system for one component of motion is 12½ grains, or about 1/3200 part of that of the pendulum.

Anticipating, from the work of Mr. Prince, of the Transit Commission in the Times Square case of the New York Subway, that the period of vibrations set up by trains would be of the order of 20 or more per sec., all moving parts were kept as light in weight as was consistent with rigidity to resist deformation of the individual parts at high frequencies.

Record was taken of vibrations set up in the surface of the earth on the cultivated earth floor inside the greenhouse. Oak stakes 3 in. square were driven 36 in. into the ground with the tops a few inches above the surface, and the earth was thoroughly compacted about the stakes with a sledge hammer. Three "set-ups" were made as shown in Fig. 20. The records cover the passing of fifty-eight trains of all classes—passenger, freight, milk, and combination—in both directions, east and west; and they include five days of operation.

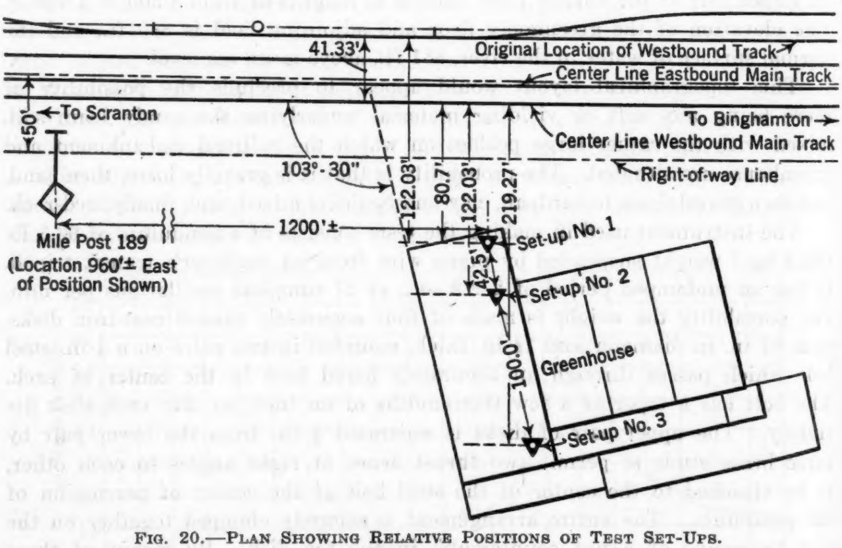

FIG. 20.—PLAN SHOWING RELATIVE POSITIONS OF TEST SET-UPS.

Set-Up No. 2 was assumed to represent the condition existing before the track was relocated; Sept-Up No. 1, the site of present maximum disturbance; and Set-Up No. 3 was located as far from No. 1 as it was possible for it to be and still remain within the greenhouse.

The method of mounting the seismograph on stakes driven in the ground was adopted to harmonize with the type of construction used for the greenhouse which consists of a glass roof supported on iron pipe posts resting on short wooden fence posts driven in the ground flush with the surface.

The maximum amplitude recorded for each train movement at each set-up was taken, for comparison, as representing a measure of the maximum earth movement to which the column footings in that vicinity were subject.

Interpretation and Tabulation of Results.—In any seismograph constructed on the pendulum principle with a system of multiplying levers, the actual amplitude of the earth's motion may be calculated from the amplitude of the trace on the seismogram by means of the formula:

$$A_e = \frac{A'}{V_0}\sqrt{\left(1 + \frac{T_e^2}{T_0^2}\right)^2 - 4\,\frac{\dfrac{T_e^2}{T_0^2}}{1 + \dfrac{1}{\pi^2}\,(\log.\ \rho)^2}} = \frac{A'}{V_0}\,U \quad \ldots\ldots\ldots (7)$$

in which,

A_e = the amplitude of the earth's motion from zero position;
A' = the amplitude of the trace record from zero position;
T_e = the period of the earth movement, in seconds;
T_0 = the period of the undamped pendulum of the seismograph;
V_0 = the magnification constant of the seismograph; and,
ρ = the damping ratio.

From the records the number of complete earth vibrations per second appears to vary from 15 to 25, or an average of, say, 20; which gives for T_e a value of $\frac{1}{20}$, or 0.05 sec. The period of the undamped pendulum is 54 swings, or 27 complete oscillations per min., which gives for T_0 a value of $\frac{60}{27}$, or 2.222+ sec. The ratio of the two periods, then, is, $\frac{T_e}{T_0} = \frac{0.05}{2.222+} = 0.0225$.

The magnification constant of the seismograph, V_0, is obtained from a consideration of the ratio of the lever arms: The vertical lever with a ratio of 5 in. to 1 in. and the recording arm with a ratio of $14\frac{1}{2}$ in. to $\frac{1}{2}$ in. For the complete lever system there is a magnification of $\frac{5}{1} \times \frac{29.7}{1} = 148.5$.

To obtain the damping ratio, ρ, the pendulum weight is gradually pushed to one side and released, producing on the record a curve similar to Fig. 21.

The ratio of the amplitude, y_1, to the amplitude of y_2, or $\frac{y_1}{y_2}$, is the damping ratio which equals 3 for the instrument under consideration as adjusted in these tests.

A collineation diagram, Fig. 22(*a*), has been drawn by F. W. Sohon,* for a graphical solution of Equation (7), by which the value of the radical factor modifying the value, $\frac{A'}{V_0}$, and usually designated as U, may be readily obtained. This diagram consists of a circle of unit radius calibrated for the period ratio, $\frac{T_e}{T_0}$, a tangent line calibrated for values of the damping ratio, ρ, and a secant line calibrated for the value of the radical factor, U.

* See *Bulletin*, Seismological Soc. of America, Vol. XIV, No. 3, September, 1924.

Laying a straight-edge from the point on the circle indicated by the ratio of $\frac{T_e}{T_0} = 0.0225$, which is very near the point of "sudden shock" at the junction of the circle with the secant line, to the point on the tangent indicated by the damping ratio of 3, the value of U is read at the intersecting point on the secant line.

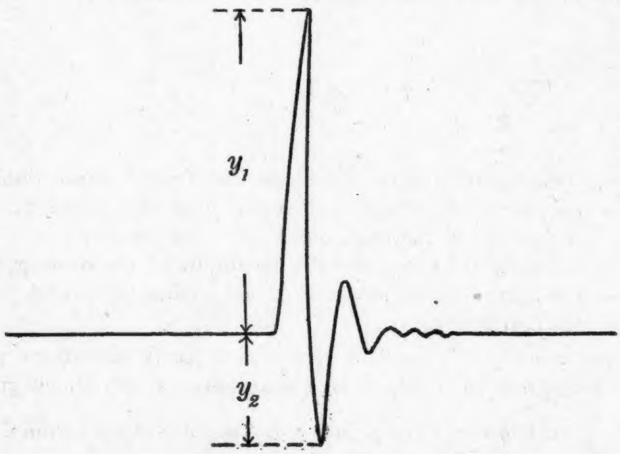

FIG. 21.—CHARACTERISTIC CURVE PRODUCED BY A PENDULUM TO DETERMINE THE DAMPING RATIO.

The diagram (Fig. 22), however, is on such a small scale that it is not convenient to use where values of $\frac{T_e}{T_0}$ are less than 0.1, or where earth periods, T_e, exceed 4 or 5 per sec.; therefore, the writer has computed and calibrated a small portion of this diagram in the vicinity of the sudden shock point, using a radius more than sixty-seven times as large as that in Fig. 22(a) (see Fig. 22(b)). The direction of the line from u to ρ can be obtained on Fig. 22(a) and a line parallel thereto can then be drawn from the proper point on the enlarged circle (Fig. 22(b)) to intersect the secant line for values of U as indicated by the dashed line. The enlarged circle has been calibrated for earth periods, T_e, running from 3 to 25 per sec., as well as for values of u or $\frac{T_e}{T_0}$ in the neighborhood of 0.1.

To show the relation of the smaller diagram, the dashed line, drawn parallel with the line from $u = 0.0225$ to $\rho = 3$, is drawn from $T_e = 20$, and cuts the secant for values of U at 0.9996 or 0.0004 less than unity. This is so near to unity that the error involved in using unity is less than that which is unavoidably involved in scaling the amplitude on the record, and this value of unity, therefore, has been used in reducing the results of the writer's tests to actual amplitude of the earth's motion.

In reducing the results of the tests, the maximum recorded double amplitude or displacement, to both sides of the zero position, shown for each train

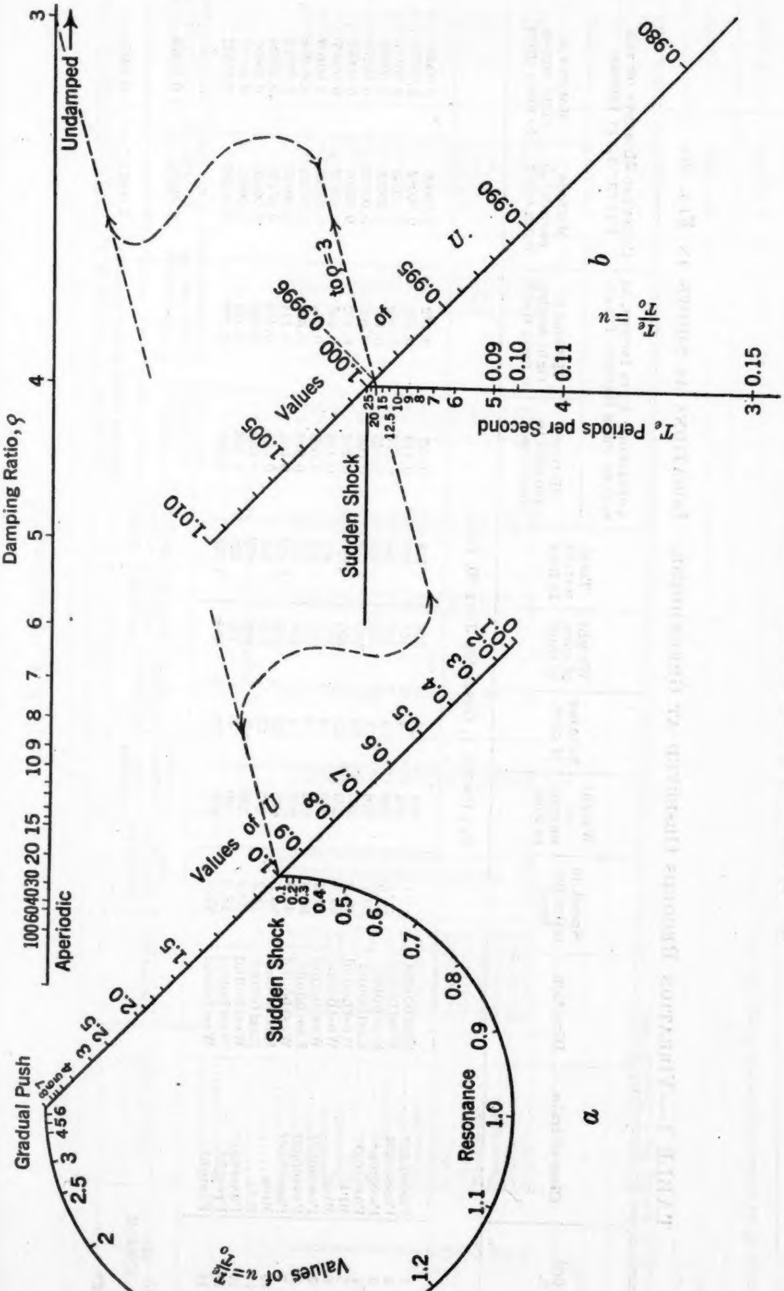

FIG. 22.—DIAGRAM FOR THE SOLUTION OF EQUATION (7).

TABLE 1.—VIBRATION RECORDS OBSERVED AT GREENHOUSE. LOCATIONS AS SHOWN IN FIG. 20.

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	AMPLITUDE, A', IN INCHES, AS SCALED FROM RECORD TRACE.		COMPUTED MOMENTS OF THE EARTH, A_e, IN INCHES.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
SET-UP No. 1; OBSERVED JUNE 20, 1925.											
1	Passenger.........	Eastbound	....	242	10	536	778	0.097	0.132	0.00066	0.00091
2	Passenger.........	Westbound	....	224	4	161	385	0.119	0.335	0.00082	0.00231
3	Passenger.........	Eastbound	53.0	226	3	170	396	0.077	0.155	0.00053	0.00107
4	Passenger.........	Eastbound	20.7	226	7	242	468	0.045	0.139	0.00031	0.00096
5	Milk..............	Westbound	40.0	235	26	817	1 082	0.090	0.403	0.00062	0.00278
6	Milk..............	Westbound	60.0	235	22	692	927	0.292	0.372	0.00201	0.00256
7	Passenger.........	Westbound	31.6	240	6	199	439	0.152	0.446	0.00105	0.00307
8	Passenger.........	Eastbound	36.0	242	9	561	803	0.086	0.202	0.00059	0.00139
9	Passenger.........	Westbound	42.9	225	4	208	433	0.092	0.383	0.00063	0.00264
10	Milk..............	Eastbound	30.0	235	25	817	1 052	0.066	0.114	0.00045	0.00078
11	Milk..............	Eastbound	13.3	235	22	710	945	0.259	0.470	0.00178	0.00324
12	Passenger.........	Westbound	60.0	242	10	628	870	0.058	0.083	0.00040	0.00057
13	Freight...........	Westbound	18.0	150	6	170	320	0.053	0.161	0.00036	0.00111
14	Freight...........	Westbound	24.0	240	35	865	1 105	0.083	0.176	0.00057	0.00121
Totals..........										0.01078	0.02460
Average..........										0.00077	0.00176

TABLE 1.—(*Continued.*)

TABLE 1.—(*Continued.*)

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	Amplitude, A', in Inches, as Scaled from Record Trace.		Computed Moments of the Earth, A_e, in Inches.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
				Set-Up No. 1; Observed June 23, 1925.							
1	Passenger.........	Eastbound	43.9	242	9	531	773	0.173	0.220	0.00119	0.00152
2	Passenger.........	Westbound	48.6	235	4	165	400	0.186	0.282	0.00094	0.00194
3	Passenger.........	Eastbound	41.9	226	5	291	517	0.090	0.171	0.00061	0.00118
4	Passenger.........	Eastbound	18.5	235	6	203	438	0.071	0.114	0.00049	0.00079
5	Milk..............	Westbound	36.0	235	26	849	1 084	0.202	0.431	0.00139	0.00297
6	Milk..............	Westbound	34.6	240	23	714	954	0.222	0.314	0.00153	0.00216
7	Combination.......	Westbound	50.0	242	6	193	435	0.127	0.242	0.00088	0.00167
8	Passenger.........	Eastbound	39.1	240	9	576	816	0.178	0.197	0.00123	0.00136
9	Passenger.........	Westbound	40.9	225	3	158	383	0.188	0.287	0.00129	0.00198
10	Milk..............	Eastbound	30.0	235	27	886	1 121	0.256	0.235	0.00176	0.00162
11	Milk..............	Eastbound	27.7	240	21	687	927	0.138	0.102	0.00095	0.00170
12	Passenger.........	Westbound	50.0	242	8	511	753	0.349	0.418	0.00291	0.00288
13*	Passenger.........	Westbound	51.4	242	4	160	402	0.257	0.393	0.00177	0.00271
14*	Passenger.........	Westbound	45.0	242	9	569	811	0.179	0.290	0.00123	0.00200
15	Light.............	Eastbound	....	250	..	.....	250	0.084	0.143	0.00058	0.00099
16	Freight...........	Westbound	Stopping	228	53	1 860	2 088	0.220	0.439	0.00151	0.00303
Totals........										0.01976	0.02950
Average of Set-Up No. 1, June 23, 1925........										= 0.00124	0.00184
Average of Set-Up No. 1, June 20, 1925........										= 0.00077	0.00176
Total........										= 0.00201	0.00360
Average of all trains recorded at Set-Up No. 1........										= 0.00101	0.00180

* Record taken June 22, 1925.

TABLE 1.—(*Continued.*)

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	Amplitude, A', in Inches as Scaled from Record Trace.		Computed Moments of the Earth, A_e, in Inches.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
				Set-Up No. 2; Observed June 21, 1925							
1	Passenger........	Eastbound	42.8	242	10	531	773	0.061	0.121	0.00042	0.00088
2	Passenger........	Westbound	54.5	235	4	150	385	0.141	0.296	0.00097	0.00204
4	Passenger........	Eastbound	38.7	225	6	193	418	0.076	0.102	0.00052	0.00070
5	Milk..............	Westbound	50.0	235	26	847	1 082	0.109	0.154	0.00075	0.00106
6	Milk..............	Westbound	24 0	235	22	690	925	0.040	0.063	0.00028	0.00043
7	Combination......	Westbound	30.0	224	5	144	368	0.105	0.267	0.00072	0.00184
8	Passenger........	Eastbound	39.1	242	10	636	878	0.059	0.106	0.00041	0.00073
10	Milk..............	Eastbound	33.3	235	26	843	1 078	0.043	0.063	0.00030	0.00043
11	Milk..............	Eastbound	32.7	235	21	688	923	0.066	0.171	0.00045	0.00118
12	Passenger........	Westbound	46.2	242	10	624	866	0.134	0.284	0.00092	0.00196
15	Freight...........	Westbound	....	228	57	1 715	1 943	0.143	0.297	0.00099	0.00205
16*	Freight...........	Eastbound	15.3	287	68	2 936	3 223	0.052	0.120	0.00036	0.00083
17	Freight...........	Eastbound	....	287	69	2 953	3 240	0.067	0.096	0.00046	0.00066
Totals..........										0.00755	0.01464
Average..........										0.00054	0.00105

*Less than one-half of train on record.

TABLE 1.—(*Continued.*)

Record No.	Class of train.	Direction.	Speed, in miles per hour.	Weight of engine, in tons.	Number of cars.	Weight of cars, in tons.	Total weight, in tons.	Amplitude, A', in Inches as Scaled from Record Trace.		Computed Moments of the Earth, A_e, in Inches.	
								Motions parallel to main track.	Motions at right angles to main track.	Motions parallel to main track.	Motions at right angles to main track.
Set-Up No. 3; Observed June 22, 1925											
2	Passenger.........	Westbound	52.9	241	4	155	396	0.059	0.082	0.00041	0.00056
3	Passenger.........	Eastbound	48.6	226	3	170	396	0.056	0.053	0.00037	0.00036
4	Passenger.........	Eastbound	18.2	235	6	2 000	435	0.035	0.033	0.00024	0.00023
5*	Milk	Westbound	30.0	241	26	843	1 084	0.084	0.087	0.00058	0.00060
6	Milk	Westbound	29.5	241	24	747	988	0.080	0.118	0.00055	0.00081
7	Combination.......	Westbound	43.9	242	6	197	439	0.063	0.064	0.00043	0.00044
8	Passenger.........	Eastbound	34.6	242	9	577	819	0.055	0.045	0.00038	0.00031
9	Passenger.........	Westbound	37.5	225	4	210	435	0.050	0.065	0.00034	0.00045
10	Milk	Eastbound	26.1	241	25	817	1 058	0.030	0.036	0.00021	0.00025
11	Milk	Eastbound	25.3	241	21	687	928	0.033	0.043	0.00023	0.00030
12	Passenger.........	Westbound	52.9	242	8	511	753	0.049	0.238	0.00034	0.00161
15	Freight...........	Eastbound	14.5	228	40	1 539	1 767	0.037	0.038	0.00025	0.00026
Totals..........										0.00433	0.00618
Average..........										0.00036	0.000515

*Coasting, not using steam.

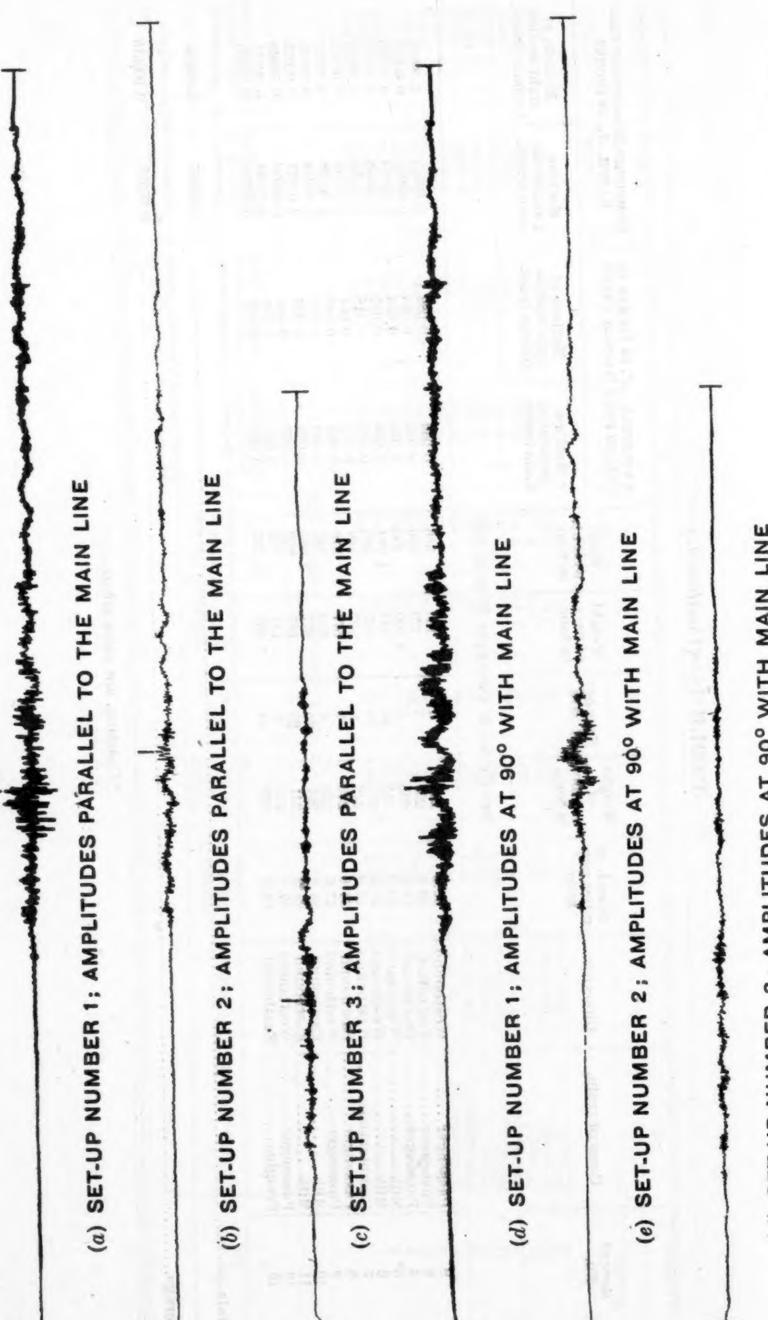

Fig. 23.—Comparison of Traces for Record No. 8, Showing Amplitudes Parallel and at Right Angles to the Main Line Tracks.

and for both horizontal components of motion, parallel with and at right angles to the track, is measured to the nearest 0.001 in. by means of a micrometer caliper gauge. The distance so obtained is divided by 2 to get the amplitude from the zero position, and this amplitude, A', divided by V_0, or 148.6, gives the actual amplitude of earth movement from zero position. This computation has been carried out to the fifth decimal place, the result being in hundred thousandths of an inch.

Table 1 illustrates the general arrangement of observed data obtained during these tests. Identical record numbers apply to identical trains for each set-up. For example, in Fig. 23, Record No. 8 (Table 1) refers to the amplitudes observed during the eastbound passage of the Lackawanna Railroad Passenger Train No. 6 as determined on successive days. As reproduced in the paper, the trace is magnified 119.6 times the original vibration; the magnification of the original graph was 148.5.

The maximum amplitude of earth movement from zero position, A_e, is found usually, but not invariably, at the time the locomotive is opposite the instrument, the maximum parallel with the track seldom occurring at the same instant as that at right angles thereto. Motion at right angles to the track is generally, but not always, considerably greater than that parallel with the track (see Table 1).

A comparison of maximum amplitudes given in Table 1 shows that the westbound trains produce a movement nearly twice that of the eastbound trains. This damping of the vibrations from the eastbound track seems to be due to the weight of the 7-ft. cinder fill, ballast, and track of the westbound main track under which the vibrations passed to reach the seismograph.

Conclusion.—In conclusion, if a profile is drawn on a line extending from the tracks through the greenhouse and passing through the three observation points (see Fig. 20), using the average of all the maximum observed amplitudes at right angles to the track at each set-up as elevations above this line, it will be possible to visualize the relation between amplitude of the earth's motion and distance from the track. Such a profile is illustrated by Curve *A* in Fig. 24.

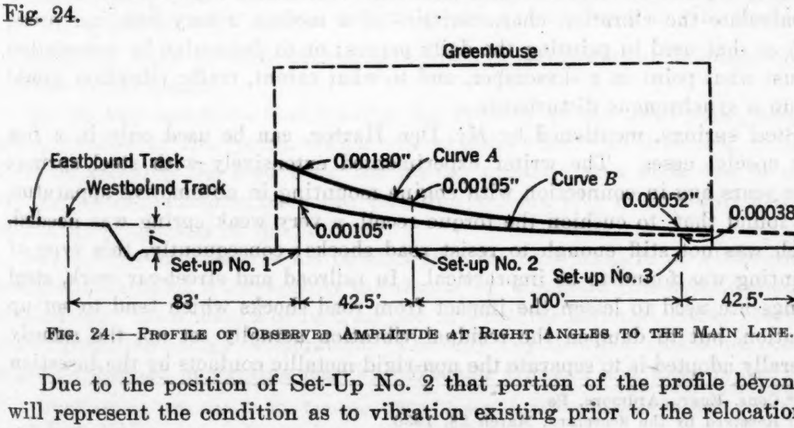

FIG. 24.—PROFILE OF OBSERVED AMPLITUDE AT RIGHT ANGLES TO THE MAIN LINE.

Due to the position of Set-Up No. 2 that portion of the profile beyond it will represent the condition as to vibration existing prior to the relocation of the main tracks 41.33 ft. closer to the greenhouse.

If this part of the profile with a northward extension of 42.5 ft., based on the indicated decrease in amplitude with distance, is moved that distance south within the first profile so that Set-Up No. 2 coincides with Set-Up No. 1, then the difference between the gradient lines of the two profiles (Curve *A* minus Curve *B*) will represent the increase in amplitude affecting the greenhouse due to the shift in position of the main tracks.

The base line of the profile makes a northeasterly angle of 103° 30′ with the center line of the westbound main track, and is parallel with the center line of the greenhouse. The distances to the several points along this line from the center line of the westbound main track are, therefore, correspondingly greater than the right-angle distances to the same points as given in Fig. 20.

A record was also taken of an Erie Railroad freight train passing on the opposite side of the Susquehanna River, at a distance of about ¾ mile from Set-Up No. 1. The maximum observed amplitude parallel with the track was 0.00013 in. and that at right angles, 0.00011 in.

Time marks on the records were spaced 10 sec. apart. The instant when the locomotive was opposite the seismograph is indicated in Fig. 23 by a free-hand mark above the record. It was observed that vibrations began from 4 to 6 sec. before this instant.

STEPHEN E. SLOCUM,* M. AM. SOC. C. E. (by letter).†—In the discussion of this paper by Mr. Den Hartog‡ the point is made that there is a rational explanation of the phenomena accompanying these tests, which of course is the case. There is a wide distinction, however, between a qualitative explanation and a quantitative measurement, and in actual practice it is undeniably true that the determination of nodes, natural frequencies, etc., is, in general, best made by experiment. The idea which the writer intended to convey was that even in the simplest cases it is necessary to make certain assumptions as to uniformity of material and conditions of constraint before a numerical solution of the vibration characteristics can be attempted. In most actual structures, the constraints are so involved as to make any attempt at a theoretical solution impossible. For instance, it would seem to be a hopeless task actually to calculate the vibration characteristics of a modern rotary printing press, such as that used in printing the daily papers; or to determine by calculation at just what point in a skyscraper, and to what extent, traffic vibration would set up a synchronous disturbance.

Steel springs, mentioned by Mr. Den Hartog, can be used only in a few very special cases. The writer experimented extensively with steel springs some years ago in connection with engine mounting in automotive apparatus, and found that, to cushion the torque recoil, a very weak spring was needed, which was not stiff enough to resist road shocks; consequently, this type of mounting was found to be impractical. In railroad and street-car work, steel springs are used to lessen the impact from road shocks which tend to set up vibration, but to dampen the residual vibration actually set up, the remedy generally adopted is to separate the non-rigid metallic contacts by the insertion

* Cons. Engr., Ardmore, Pa.

† Received by the Secretary, March 28, 1930.

‡ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 153.

of pads of resilient damping material. For the special purpose for which this research was undertaken, namely, the determination of the physical properties and vibration characteristics of damping materials suitable for use as isolation pads under grillage foundations of tall buildings, steel springs are obviously out of the question.

In Mr. Tatnall's discussion,* the principal point made is that change in design is a more effective remedy for vibration than isolation. This is true only of machinery built from a stock design and operated under fixed and uniform conditions. For instance, the crank shaft of a Diesel engine should be designed so as to be free from excessive torsional vibration at the operating speed of the motor. In machinery like compressors and blowers, however, it is usually impractical to eliminate vibration by changes in design, and the only feasible method of preventing the transmission of vibration is isolation from adjacent structures.

Mr. Dawley's criticism† of the author's experimental set-up would be pertinent and valuable if the experiments had been intended as a basis for mathematical analysis of the compound system, although his suggestions by no means cover all the difficulties involved. As this series of experiments was apparently the first of its kind, at least so far as published results are concerned, there were numerous fundamental questions to be answered before the problem could be stated in sufficiently exact terms to warrant an elaboration of set-up which would differentiate results more closely.

For instance, some of the primary questions involved and on which no information could be gained elsewhere, were such as the following:

(*a*) For a substance like asbestos, what effect does increasing the unit load on the material have on its properties as a vibration damper?

(*b*) For lateral insulation of footings would a mastic joint be as effective as corkboard, or if the latter was used, would a layer 4 in. or more in thickness be necessary, or would a 1-in. layer be sufficient?

(*c*) For track insulation, would a layer of sand be more effective than a layer of gravel? This question was of prime importance, since if experiment had indicated that sand was greatly superior, its use would have involved excavating a layer of natural glacial drift, easily plowed up into gravel, and replacing it with sand.

Evidently none of these problems involved a mathematical determination of nodes or harmonics.

Mr. Dawley mentions that loading the H-beam at the third point would create a node at this point. This was exactly what was intended, as the effect of applying the load from the testing machine at a node was to cut out the testing machine from the vibrating system, and to just this extent reduce the complexity of the problem.

Mr. Dawley expresses as his opinion that the best measure of a vibration damper or insulator is its capacity to "absorb" or "dissipate" the kinetic energy of vibration. As a matter of fact, no pad of whatever material, either absorbs or dissipates the energy of vibration. The only way in which this could occur would be through generating heat by molecular friction, and there is not the

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 156.

† See p. 1177.

slightest evidence that anything of the kind occurs. The word which most fully describes a vibration damper is "isolation". The effect of such isolation is to short-circuit the vibration, reflecting a certain fraction of its energy back into the transmitting system; and to just the extent to which it is effective in reflecting the energy of vibration, it is successful in preventing its transmission to the receiving system.

To handle the vibration problem intelligently requires considerable versatility of method. In the case of machinery, the first consideration is whether change in design is possible, and if so, whether it will reduce vibration to a point where it is not objectionable. For heavy machinery, the design of the foundation is often involved in the problem. In the case of traffic vibration, the street pavement or track roadbed is an important factor to be considered. In tall buildings, an important remedy lies in isolation of the structure by various means. A notable consequence of the tests summarized in this paper was the discovery that a simple compressive pad was by no means the most effective method of applying resilient dampers. The tests also disclosed the fact that the range of damping materials commercially available is extremely limited, and that none possesses all the required properties of an effective damper or isolation medium under all conditions of use and loading.

As Mr. Tatnall points out, vibration engineering should be a fertile field for the activities of the Society. Although there seems to be many specialists working independently in this field along such widely divergent lines as seismology in oil fields, and impact stresses in bridges, there is no attempt to co-ordinate their efforts or systematize their results. In London, England, the serious effects of traffic vibration on such notable structures as the Nelson Column, the National Gallery, in Trafalgar Square, St. Paul's Cathedral, and Westminster Abbey, has forced the question of remedying matters on the attention of the Office of Works, and this has made the question of vibration a permanent subject of research and discussion by the Science Committee of the Royal Institute of British Architects.

In Germany, the Scientific Council of the Vereins Deutscher Ingenieure took steps as early as 1923 to foster research in vibration engineering, and on December 11, 1924, that Society organized a Committee on Vibration. In order to give direction to this research, the Committee called attention to three specific problems, namely, dynamic strain in machines and structures due to high frequency vibrations; general disturbance, such as noise and tremor due to vibration; and research in the technic of mechanical vibration, both in scientific laboratories and in industry, with the related problem of the endurance of materials of construction subject to vibration.

In 1927 this Committee on Vibration held a two-day convention at Braunschweig. The principal topics discussed at this conference were approved methods of conducting vibration experiments; laboratory methods in current use; necessary and sufficient conditions for judging the value of vibration measurements; energy absorption or damping capacity of building materials; acoustic experiments with various materials and types of construction; structure of crystals as related to vibration endurance; and electrical apparatus for measuring the strength of sound.

In 1928, this Committee held a three-day session in Darmstadt, the principal topics considered being the fatigue of materials of construction subject to vibration; experimental measurement of vibration in railway tracks and bridges; natural modes of vibration of machine foundations; destructive effects of vibration due to street traffic as related to condition of pavement and type of tire; development of instrumental methods of measuring vibration; absorption of energy of vibration by materials of construction; and the use of annealed instead of hardened steel for machine springs.

For the first time, in the 1927 Edition of "Hütte", a well-known handbook for German engineers, a section was devoted to the subject of vibration.

Heretofore, the problem of vibration has been confined in its engineering aspects almost entirely to machine design and operation. In this country the rapid increase in height of skyscrapers, together with a growing volume of street and subway traffic, has created a new and vital vibration problem. The rapid growth of this field in other lines, such as radio engineering and seismographic prospecting for oil on the Gulf Coast, has given a new aspect to the entire question. It has long been recognized that vibration is a basic phenomenon in physics and chemistry. It is equally apparent that it is a basic principle in many branches of technology, such as electrical engineering, sound reproduction, and geo-physics. The particular phase which commands immediate attention is the destructive effects of vibration on men and materials. In this particular line of effort, perhaps greater advances have been made in this country than elsewhere, but it still remains for American engineers to co-ordinate this accumulated knowledge and to systematize the principles of vibration control into a specialized branch of engineering. The Society is the obvious agency for this purpose.

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MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM BROWN, M. Am. Soc. C. E.*

DIED DECEMBER 24, 1929.

William Brown was born in Glasgow, Scotland, in 1850, and was educated at Renfrew and Paisley Grammar Schools and at the University of his native town. He was the son of Andrew Brown, who invented and patented the type of vessel now known as a "hopper dredge". This design was improved in later years by the introduction of the "stern well type" which also was patented by Mr. Andrew Brown.

Beginning his apprenticeship in 1866 in the Engineering Department of Messrs. William Simons and Company of Renfrew, William Brown was assumed with his brother, Col. Walter Brown, as a partner by his father. He was associated with this firm, which was converted into a Limited Liability Company in 1900, until his death on December 24, 1929.

Mr. Brown was appointed Assistant Manager of the Engineering Department in 1872 and Manager in 1876. He was taken into partnership in 1888, and was elected a Director in the Company when it was floated. He then became Chairman and Managing Director in 1907, which position he held until his death.

He identified himself very closely with the development of all types of marine dredging plant, particularly suction dredges and in the adaptation of this type of dredge for working in clay or similar materials. He patented a number of devices calculated to increase the efficiency of such machinery. The large fleet of dredging plant originally used at Durban, Natal, for the improvement of the depths in the bar and harbor there, and a large number of very powerful dredges which are in use at various ports in India, were designed under his direction.

Mr. Brown was a Member of the Institution of Civil Engineers, the Institution of Naval Architects, the Institution of Mechanical Engineers, and the Institution of Engineers and Shipbuilders in Scotland. At various times, he contributed to the *Proceedings* of the first and last named. He also found time to devote himself very largely to the public interests of Renfrew, and he represented the Renfrew Town Council on the Clyde Navigation Trust for a great number of years.

In 1918 Mr. Brown had the honor of Commander of the British Empire conferred on him in recognition of his services during the World War; and, in 1923, the Burgh of Renfrew further honored him by presenting him with the Freedom of the Burgh.

Mr. Brown was elected a Member of the American Society of Civil Engineers on October 3, 1911.

* Memoir prepared by Wm. Simons & Co., Ltd., Glasgow, Scotland.

CHARLES LINCOLN CARPENTER, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1929.

Charles Lincoln Carpenter, the son of the Rev. Charles Carrol and Feronia (Rice) Carpenter, was born at Amherst, Mass., on June 17, 1867. On both sides of the family there was a background of sturdy New England stock, identified with the civil, religious, and martial life of the Colonies from their beginning. The first of the name in this country was William Carpenter, an English Puritan, who settled in Weymouth, Mass., in 1638. On the maternal side he was descended from Deacon Edmund Rice, an early settler (1638) of Worcester, Mass., and on the same side he was allied to the Knowlton and Pomeroy families.

Mr. Carpenter was graduated from Dartmouth College in 1887 and from the Thayer School of Civil Engineering in 1889. The funds for this college education were largely earned by himself in doing odd jobs around the campus and in working during the summer vacations.

After his graduation, Mr. Carpenter went to Nicaragua where he was engaged for two years for the Canal Construction Company as Leveler and Transitman on preliminary and railroad location surveys. He was in charge of 30 miles of telegraph line construction; detail surveys for locks and embankment of canal; and also of surveys on construction and improvements of Greytown Harbor.

In July, 1891, he returned to Massachusetts where he was employed with the Boston Board of Survey and the Street Commission of Boston for seven years, one year as Transitman and six years as Engineer in charge of surveys, calculations, and the laying out of a new street system in Boston.

In May, 1898, Mr. Carpenter went to Alaska on a prospecting and mining trip. On his return in November, 1900, he spent several months on preliminary and location surveys for the Boston and Worcester Electric Railroad Company, and then accepted a position in Cuba on railroad location and construction with the Cuba Central Railroad Company as Transitman and Assistant Division Engineer. At the expiration of a year in Cuba, he returned to the United States and was engaged for about three months as Assistant to the City Engineer of Gloucester, Mass. In September, 1902, however, he commenced work with the Federal Government as U. S. Junior Engineer on river and harbor work at Boston.

When work on the Panama Canal was begun, Mr. Carpenter took one of the first parties to Panama in June, 1904, and the necessary surveys of the Great Gatun Lake Basin were started. Possible dam sites at Gamboa and Althajucta on the Chagres River, were investigated and reported on; and surveys for the relocation of the Panama Railroad at Bas Obispo, the location of the center line of the Canal on the Atlantic end, with the necessary borings, and the location of the Canal Zone boundary line, were all done under his supervision during the first two years.

* Memoir prepared by E. F. Sinz, Assoc. M. Am. Soc. C. E.

In July, 1906, Mr. Carpenter was made Resident Engineer of the Gatun Dam and in July, 1907, Resident Engineer of the Gatun Locks, which position he held until July, 1908. All clearing of the site of this huge earth dam and these locks, surveys, borings for foundation, and preliminary excavations were made while he was in charge, and his work as an organizer was outstanding. The Porto Bello Quarry, where all the rock for the immense concrete structures on the "Atlantic end" was quarried and crushed, was put in operation under his direction.

In September, 1908, Mr. Carpenter again went to Cuba, this time for J. G. White and Company, in charge of the reconstruction of the Cuban Eastern Railroad. During the year of this contract, the road-bed was rebuilt so that it would stand up under traffic during the tropical rains and floods. He continued with this railroad, which was re-organized and named the Guantanamo and Western Railroad, as General Manager and built up the property into a prosperous organization.

In December, 1911, Mr. Carpenter went to Porto Rico as Superintendent of the Ponce and Guayama Railroad. In June, 1912, he was appointed to fill the vacancy of General Manager of the Central Aguirre Sugar Company and of the Ponce and Guayama Railroad Company. Later, the duties of Vice-President of these two companies were added as well as those of Vice-President and General Manager of The Central Machete Company and of the Santa Isabel Sugar Company. In 1919, he became one of the Managing Partners of Luce and Company, a land-holding organization in Porto Rico. He held all these positions until his death.

The professional career of Mr. Carpenter was a varied one and took him to many parts of the North American Continent—from Alaska to Panama. He was an outstanding administrator, as the progress of the works that he directed will testify. He demanded results, but he never asked his men to go where he would not go himself. During his work in the tropical jungles, he never spared himself, and this willingness to share all hardships won for him the loyalty and friendship of his associates, many of whom were always glad to work under him, wherever he happened to be. He was a great lover of the outdoors and his favorite form of vacation was a tramp through the New England mountains or a canoe trip along the rivers of Canada.

Death came to Mr. Carpenter very suddenly, while he was on one of these canoe trips in one of the wildest parts of the Upper Gatineau River region of Canada. A few days of hard paddling with several difficult portages were too much for his strength, and he died of heart failure on September 28, 1929. He will long be remembered by his associates as an able, beloved executive and by the poor as a generous giver when help was needed. Mr. Carpenter was a Mason, and a member of various geographical and historical societies and of the Appalachian Mountain Club. He had contributed to the Byrd Antarctic Expedition and to many associations for the preservation of forests and of wild life. He cheerfully gave to the poor and distressed, particularly to children. Those who knew him well loved him for this whole-hearted friendship and kindness.

Mr. Carpenter was married on December 15, 1892, to Charlotte Florence Sullivan, of Boston, Mass., who, with three children, James Sullivan, Thomas Rice, and Charles Carrol, survives him.

Mr. Carpenter was elected a Member of the American Society of Civil Engineers on December 6, 1905.

ERNEST LESTER JONES, M. Am. Soc. C. E.*

DIED APRIL 9, 1929.

Ernest Lester Jones, the son of Charles Hopkins and Ida (Lester) Jones, was born at East Orange, N. J., on April 14, 1876. He received his educational training at the High School in Orange, N. J., and at Newark Academy. Later he matriculated at Princeton University in the Class of 1898, from which institution he received the Bachelor of Arts Degree.

Following the completion of his studies at Princeton University, Mr. Jones was engaged in research, secretarial work, and business for a number of years. Early in 1913 he entered the service of the Federal Government, President Wilson having appointed him Deputy Commissioner in the United States Bureau of Fisheries. He remained in this position until April 14, 1915, when he became Superintendent (title changed to Director in 1919) of the United States Coast and Geodetic Survey, which position he held until his death.

During his residence in Washington, D. C., Colonel Jones served in the District of Columbia Militia, from Private to Major. During a portion of 1918 and 1919 he was on furlough from the Coast and Geodetic Survey, and was commissioned a Lieutenant-Colonel, U. S. Signal Corps. Later, he became Colonel, Division of Military Aeronautics, and served with the American forces in France in the World War. For exceptional services during the war period he was decorated by the King of Italy as Officer of the Order of S. S. Maurizio and Lazzaro, and Fatigue de Guerre (Italy); he was also an Officer of the Legion of Honor (France). Immediately following the cessation of the war, when men's minds everywhere were turned toward matters of rehabilitation, Colonel Jones was among the first to consider the welfare of those who had been at the battle front in Europe, and thus it came about that he was the organizer of the first post of the American Legion (George Washington Post, Washington, D. C.), and also an organizer and incorporator of the National Legion.

As the efficient administrator of a Federal Bureau, Colonel Jones early came to see the need for better conditions and more adequate salaries for employees in the Federal Service, and his advocacy of their cause in bringing these urgent needs to the attention of the proper authorities had an important part in securing remedial legislation. The great improvement in the efficiency of the personnel of the Federal Service, as a result of this legislation, has amply justified the wisdom of his efforts.

* Memoir prepared by R. L. Faris, M. Am. Soc. C. E.

It was a part of his philosophy of human affairs that the best work can be done only when men have the best tools and appliances for doing it, and so it was among his basic endeavors while Director of the Coast and Geodetic Survey that the Bureau's engineers be supplied with adequate ships and modern instruments and equipment. These things he achieved in a large measure, the good effects of which are reflected in a larger and better volume of work and a finer spirit of performance by the entire personnel, so that the Bureau now meets the purposes of its being with a growing satisfaction and increased efficiency.

Throughout his administration of the Bureau, Colonel Jones exemplified high executive ability. He was outstanding in his loyalty to the work of the organization and to his associates and subordinates, and in turn he engendered in them such sentiments toward himself. He worked constantly for the improvement of the Service under his direction so that the public might thereby be better served. He was positive in responding personally to whatever seemed necessary to advance each class of work and always co-operated with those of his associates who were making progress. He spared himself no amount of effort and toil to attain the things he thought were right and needful to be done. He had a humane and sincere sympathy for all who requested his assistance. Although firm in his opinions, yet he was considerate of the views of others. He was loyal alike to those whom he served and to those who served under him, and also to his own promises and obligations in that he gave the best that was in him in all his endeavors.

In addition to his duties as Director of the Coast and Geodetic Survey, Colonel Jones was also Commissioner of the International Boundary between the United States and Canada and Alaska and Canada, from February, 1921, until his death. He was a member of the Aerial Patrol Commission of the United States, and a member of a number of Government and scientific missions, one of the last of which was as a delegate to the International Geographic Congress, at Cambridge, England, in 1928.

Colonel Jones was a member of a large number of organizations and societies, which included scientific, engineering, social, patriotic, and outdoor recreational purposes, showing thereby a wide range of active human interest and usefulness. Among the organizations of which he was an active member may be mentioned the Washington Academy of Sciences; the Philosophical Society of Washington; the American Association for the Advancement of Science; the Washington Society of Engineers; the National Geographic Society; the Meteorological Society; the American Fisheries Society; the National Association of Audubon Societies; the Society of American Military Engineers; the Military Order of the World War; and the Society of Mayflower Descendants. His membership in various clubs and civic organizations, also eloquent evidence of his wide interest in his fellow man, included the National Press Club, the Explorers Club (New York), the Aero Club of America, the Cosmos Club, and the Federal Club.

He was the author of the following Government publications: "Alaska Investigations"; "Hypsometry"; "Elements of Chart Making"; "Safeguard the Gateways of Alaska"; Earthquake Investigations in the United States"; and

"The Neglected Waters of the Pacific". In addition, he also was the author of the following (unofficial) papers: "Evolution of the National Chart"; "Science and the Earthquake Perils"; and "Aerial Surveying".

In 1919, he was granted the honorary degree of Master of Arts from Princeton University with the following citation:

"Ernest Lester Jones, Director, United States Coast and Geodetic Survey, the oldest scientific agency of our Government, writer on our coastal waterways bordering the Pacific Ocean, a resourceful administrator, increasing largely our supply of reliable maps and supervising the use of new devices for making our waters safer, notably by detecting the perilous submerged pinnacle rocks; a Colonel in the Army during the war, on active service in France and Italy, decorated by the King of Italy, awarded the Diploma of Merit by the Aerial League of America, recommended for the French Croix de Guerre; most recently instrumental in helping to form the American Legion to perpetuate American Liberty".

As a token of its personal regard for Colonel Jones, the following resolutions were adopted by the personnel of the United States Coast and Geodetic Survey and the International Boundary Commission:

Whereas, on April 9, 1929, the death of Colonel Ernest Lester Jones, Director of the United States Coast and Geodetic Survey and Commissioner of the International Boundary Commission, United States—Alaska and Canada, has deprived the nation of an earnest, fearless, and efficient public servant, who always placed devotion to duty above considerations of personal welfare; and,

Whereas, under Colonel Jones' inspiring leadership these organizations have made outstanding contributions both to the immediate public welfare and security and to the better knowledge of scientific truth on which the future of human progress in large measure depends; and,

Whereas, his deep human understanding and sympathy endeared him to all with whom he came in contact, so that those who knew him best loved him most;

Therefore, Be It Resolved, That we, for and in behalf of his friends and associates in the Coast and Geodetic Survey and the International Boundary Commission, unite in expressing our profound conviction that in the passing of Colonel Jones the Administrative Government of the United States has suffered the loss of an executive of rare ability and achievement, and that we have been deprived of a true friend and counsellor;

And Be It Further Resolved, That we extend to his bereaved family our deepest sympathy and express to them our profound sorrow and our heartfelt understanding of the great loss they have suffered.

On September 28, 1897, he was married to Virginia Brent Fox, of Louisville, Ky. He is survived by his widow and two daughters.

Colonel Jones was elected an Associate Member of the American Society of Civil Engineers on April 3, 1922, and a Member on October 12, 1925.

EDWARD STANLEY SAFFORD, M. Am. Soc. C. E.*

Died December 31, 1929.

Edward Stanley Safford was born at Boston, Mass., on September 6, 1847. Following his graduation from the English High School in Boston in 1865, he

* Memoir prepared by Louis P. Gaston, M. Am. Soc. C. E.

entered, in the same year, the Massachusetts Institute of Technology which school had just been organized. He was enrolled as a member of the first graduating class, that of 1868, and took the course in Mining and Civil Engineering.

Mr. Safford was employed, as Rodman on the first survey for the Atchison, Topeka, and Santa Fé Railroad, on July 7, 1868, from Topeka to Burlingame, Kans., a distance of 29 miles. Later, he was engaged on the long preliminary surveys, and, as Division Engineer, had charge of the construction of 45 miles of the road from Emporia to Florence, Kans.

In the fall of 1871, Mr. Safford laid out what is now the flourishing city of Hutchinson, Kans., on the Arkansas River, at the mouth of Cow Creek. He returned to Boston in the spring of 1872, and was engaged as Contractor on public works in and about Boston until 1878. He was Chief Engineer of the New London Northern Railroad Company from 1878 to 1880, after which he served as Division Engineer on the West Shore Railroad (then the New York, West Shore, and Buffalo Railroad) first at Newburgh, N. Y., in charge of construction in Orange County, New York, and, later, at the Buffalo Terminal. In 1884 and 1885, he was employed on the Philadelphia Branch of the Baltimore and Ohio Railroad, with headquarters at Upper Falls, Md. Subsequently, he established a home in Arlington, Mass. In 1885, he became Principal Assistant Engineer on the Mobile and Birmingham Railroad, remaining in Alabama from 1886 to 1888; later, he acted as Chief Engineer for the Shelby Iron Company, at Shelby, Ala.

In the winter of 1890-1891, Mr. Safford made a survey on snowshoes in Nova Scotia. From 1892 to 1895, he was Engineer in charge of construction of the Palisades Tunnel and the building of the two large coal-shipping trestles on the Hudson River, in addition to a terminal for the New York, Susquehanna, and Western Railroad Company, all of which work passed into the hands of the Erie Railroad Company soon after its completion.

In 1897, he went to Mexico and made surveys for the Chihuahua and Pacific Railroad Company from Chihuahua, Mexico, extending west for 200 km. He subsequently was chosen as Chief Engineer of this line, made location surveys, and finished the construction by May, 1900. Mr. Safford remained in Mexico until June, 1904, when he returned to the United States. He lived at Somerville, N. J., and at Sharon, Mass., for about three years, and then became Engineer for the contractors who were eliminating grade crossings on the New York, New Haven and Hartford Railroad, at New Bedford, Worcester, and Dorchester, Mass. In 1911, he went to Haiti on railroad construction. He returned to the United States the following year, thus terminating his business activity.

Mr. Safford was notable among engineers of an older generation as one who had special ability as a Locating Engineer at a time when railroad location and construction were at their height in this country. He will also be remembered by his many friends for his genial disposition and for his cheerfulness under a great affliction—deafness.

Mr. Safford was elected a Member of the American Society of Civil Engineers on December 6, 1882.

IRVING MASON WOLVERTON, M. Am. Soc. C. E.*

DIED JANUARY 6, 1930.

Irving Mason Wolverton was born at Grand Blanc, Mich., on January 29, 1869, and, in 1880, moved with his parents to Flint, Mich. After graduating from the High School at Flint, he entered the Civil Engineering Department of the University of Michigan, from which he was graduated in 1890.

After graduation Mr. Wolverton became associated with the King Bridge Company of Cleveland, Ohio, as an Engineer on structural steel work. After four years with that Company, he was appointed Chief Engineer of The New Columbus Bridge Company, of Columbus, Ohio, with which he remained until 1899.

He was then chosen Chief Engineer of The Mount Vernon Bridge Company of Mount Vernon, Ohio. He successively was made Vice-President and, in 1919, President and Treasurer, which latter position he held until his death. The Mount Vernon Bridge Company has built many notable highway and railroad structures in various parts of the United States, and, under the guidance of Mr. Wolverton, has become one of the leaders in the field of steel construction.

Death came to Mr. Wolverton very suddenly on January 6, 1930. He had gone to Columbus on a business trip and stayed over Sunday in order that he might see a physician who was to be out of town until Monday. It was thought that his trouble was only a cold. Mrs. Wolverton and several of his associates saw him on Sunday, and he seemed to be in very good condition. On Monday morning he was found dead in his bed at the hotel.

Mr. Wolverton had a reputation for loyalty to his friends and for doing well everything that he attempted. He went into all propositions carefully and in detail. His success was due to this painstaking care and to his ability to make and keep a host of friends. He was a member of the Mount Vernon Country Club and The Columbus Club, of Columbus. He was also a member of the Benevolent and Protective Order of Elks and was a Thirty-second Degree Mason.

He was married on September 12, 1893, to Florence Harriet Pope, of Allegan, Mich., and is survived by his widow, three daughters, Carlotta Schafer, of Cleveland, Ohio, Harriet Schelling, of New York, N. Y., and Frances Taylor, of Middletown, Conn., and one son, John P. Wolverton, of Akron, Ohio.

Mr. Wolverton was elected an Associate Member of the American Society of Civil Engineers on December 4, 1895, and a Member on December 1, 1903.

ADOLPHUS JAMES EDDY, Assoc. M. Am. Soc. C. E.†

DIED JUNE 28, 1929.

Adolphus James Eddy, invariably known as "Jim" Eddy, the son of Adolphus Frederick and Mary Ellen (Slover) Eddy, was born at Jacksonville,

* Memoir prepared by C. G. Conley, M. Am. Soc. C. E.

† Memoir prepared by the following Committee of the San Francisco Section, Ralph G. Wadsworth, *Chairman*, and C. A. Whitton, Members, Am. Soc. C. E., and E. A. Reinke, Assoc. M. Am. Soc. C. E.

Ore., on December 25, 1885. He completed his Grammar and High School education in Southern Oregon and moved to San Francisco, Calif., in 1904, where he worked for two years in the shops of the Union Iron Works, earning sufficient money to start his college course.

In 1906, Mr. Eddy entered the University of California and in May, 1910, was graduated with honors in Civil Engineering, Astronomy, and Military Science. He was one of the representatives of the Class selected to speak at the Commencement Exercises. For the following two years, he carried on post-graduate work at the University in Civil Engineering and allied subjects.

He began his professional career as early as 1902, when for eleven months he worked as Chairman and Instrumentman for the Southern Pacific Railroad Company. In June and July, 1908, he acted as Inspector on the construction of a highway in San Mateo County, California. After graduation from college in 1910, until 1917, and again in the early part of 1920, Mr. Eddy was on the Teaching Staff of the College of Civil Engineering at the University of California, as Instructor, Assistant Professor, and, finally, Associate Professor. During this period, he also served as Assistant Instructor in Military Science.

During the same period, Mr. Eddy devoted all his spare time to professional work, including structural designing for John Galen Howard in the summers of 1910 and 1915; for Frederick H. Meyer in 1911; for the Panama-Pacific Exposition Company in 1914; and for Charles Derleth, Jr., M. Am. Soc. C. E., in 1916.

In the fall of 1919, following his military service in the World War, Mr. Eddy was employed for three months by Fred H. Tibbetts, M. Am. Soc. C. E., on field surveys and office investigations of an irrigation project in the Sacramento Valley, involving about 5 600 acres and requiring provision for flood protection and drainage, as well as irrigation service. Commencing in May, 1920, he was employed for about three years by the Standard Oil Company of California on structural engineering work, pertaining largely to the design and construction of the Company's 22-story office building, which was one of the first high buildings constructed on or near the filled-in area of San Francisco. This work involved important and thorough investigations of foundations and complete checking of the steel frame design, special attention being given to possible earthquake effects.

In September, 1923, shortly after the inauguration of the City Manager form of government in Berkeley, Calif., Mr. Eddy was appointed City Engineer and Superintendent of Streets for that city. His work involved a wide variety of engineering problems, including street paving, sewerage systems, storm drainage, garbage disposal, concrete retaining walls, and other structures. Studies and reports were also made on Berkeley's municipal wharf, including plans for extensive repairs and additions, and preliminary studies were made for a municipal airport.

Mr. Eddy always took an extremely active interest in military work of various kinds, beginning with a two-year period of service in the Oregon National Guard in 1902-1903. At the University of California, he completed all the courses in Military Science and was graduated with the commission of

Major in the Cadet Corps. Prior to the entrance of the United States into the World War, he was active in organization of citizen-soldier activities at the University, and in the spring of 1917 he entered the first Officers Training Camp at the Presidio of San Francisco, from which he was graduated with a commission as First Lieutenant, Coast Artillery. On November 30, 1917, he was promoted to the rank of Captain, and on November 9, 1918, to that of Major. He took a command of 300 enlisted men and 4 officers, comprising an Army Artillery Park, to France and, while there, attended the Heavy Artillery School at Angers. At the time of the signing of the Armistice, he had just been ordered back to the United States in order to conduct overseas another artillery unit. In January, 1919, at Camp Eustis, Virginia, he had full charge of the test firing of an 8-in. howitzer on a self-propelled mount, and, later in that year, conducted a recruiting party through the Middle Western and Pacific Coast States. He was given an honorable discharge on November 6, 1919.

On May 4, 1921, he was appointed Captain of the 159th Infantry, California National Guard, and was rapidly advanced through the several intervening grades to the rank of Colonel, commanding the regiment, which rank he held from June 29, 1925, until his death. At the time of the disastrous fire, which swept through thirty blocks of high-class residences in Berkeley, in 1923, he risked official censure by ordering out, on his own responsibility, the local National Guard unit, and earned the gratitude of the community for the effective police and relief work accomplished under his direction.

His death, on June 28, 1929, occurred as the direct result of a major operation performed in San Francisco to cure an intestinal disorder, which had given him intermittent trouble subsequent to his service in the war.

Mr. Eddy was a member of Berkeley Lodge No. 363, F. and A. M., the Oakland Scottish Rite Bodies, and Aahmes Temple, A A. O. N. M. S. He was a Director of the Berkeley Chamber of Commerce, a member of the Rotary Club, the Faculty Club of the University of California, the American Legion, the Association of the Army of the United States, and various other societies. His scholastic honors included membership in Tau Beta Pi, Phi Beta Kappa, and Sigma Xi. His sterling character, charming personality, and boundless good humor won him the friendship and admiration of all who knew him.

He was married at Berkeley, in August, 1916, to Margaret G. Stone. They had three children, Margaret Eleanor, James Stone, and Barbara, all of whom, together with his father and mother, survive him.

Mr. Eddy was elected a Junior of the American Society of Civil Engineers on December 6, 1910, and an Associate Member on July 6, 1920.

JOHN LUCEY, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 16, 1929.

John Lucey was born in Little Falls, N. Y., on June 24, 1886, the son of the late Cornelius D. and Bridget (Lynch) Lucey.

Mr. Lucey received his early education at the public schools of Little Falls and was graduated from St. Mary's Academy, Little Falls, N. Y., in 1898. He

* Memoir prepared by John T. Collins, Assoc. M. Am. Soc. C. E.

attended Union College at Schenectady, N. Y., for a time and, later, the Engineering School of New York University from which he was graduated in 1908 with the degree of Bachelor of Science in Civil Engineering.

He began his engineering career in the Construction Department of the New York Central Railroad Company as Rodman and Transitman, making surveys and plans for the track changes in connection with the Barge Canal crossings, in September, 1908. In 1914, he was made Assistant Engineer in charge of construction of concrete arches and Office Assistant in charge of plans and final estimates of this work.

Mr. Lucey was employed by the Rockwood Sprinkler Company of New York City from May, 1914, to September, 1916, in making estimates and designing automatic sprinkler equipment.

He then returned to the New York Central Railroad Company as Assistant Engineer and, in 1916, and 1917, was in charge of the construction of a 250-ft. span, double-track, steel, truss bridge over the Barge Canal at Brewerton, N. Y., including track changes and water supply facilities.

In May, 1917, Mr. Lucey left the United States for the Dominican Republic, West Indies, to take charge as Resident Engineer, under the direction of the Department of Public Works, of harbor improvements in the Port of Puerto Plata. This work included the dredging of the harbor and the construction of a reinforced concrete pier on concrete piling with a custom house of similar material and a steel warehouse built on the pier.

For the next three years he was District Engineer, in general charge of all Departmental work in the District of the Cibao. This work consisted principally of the construction of about 200 km. of macadam highway; the surfacing and graveling of 90 km. of old dirt road; and the location, grading, and surfacing of 110 km. of new road. The greater part of this work was done by force account, Mr. Lucey being directly responsible for the organization, the direction of the work, and the handling of the funds for the payment of all labor and materials. All this was done to the entire satisfaction of the Dominican Government officials.

From 1921 to 1923 he was in the City of Mexico, Mexico, as Superintendent on the completion of a three-story concrete and steel building for the Compania Importadora del Auto Universal and on a hydro-electric survey for the Real del Monte Mining Company. From 1924 to 1926, he was with the Mexican Railroad Company as Chief of Party, under Mr. O. G. Bunsen, on the location of the proposed line between Pachuca and Tampico.

Mr. Lucey then went with the International Railways Company of Central America in Salvador, and for two years, until the end of 1928, was in charge of location and construction of the railroad for the Polichic Banana Company and as Supervising Engineer of the line between Zacapa and Ostua for the section being built under contract by Keilhaue and Rodezno.

Outside his professional life John Lucey turned to literature for his real recreation. From his childhood until his death he explored the realm of books. The telling of what he had discovered, leavened with his keen sense of humor, made of him a very enjoyable and entertaining associate. Besides his companionship, he was a true friend, always doing thoughtful things for those

whom he liked. His death was a great loss to his many friends, his engineering associates, and to his family.

He never married. He is survived by five sisters: Mrs. Mary Stewart, of Hollywood, Calif., and the Misses Katherine, Anna, Julia, and Theresa Lucey, of Yonkers, N. Y.; and one brother, Jeremiah Lucey, of Pittsfield, Mass.

Mr. Lucey was elected an Associate Member of the American Society of Civil Engineers on January 13, 1919.

CHARLES HENRY REYNOLDS, Assoc. M. Am. Soc. C. E.*

Died November 1, 1929.

Charles Henry Reynolds, the son of Mr. and Mrs. Aldrich J. Reynolds, was born in Forestville, near Manchester, Iowa, on December 3, 1874. At the age of two he moved to Fort Dodge, Iowa, where he resided until his death.

Mr. Reynolds received his education in the public schools of Fort Dodge and was graduated from the High School. After his graduation, he was Assistant to the City Engineer at Fort Dodge for five years, during which time the first pavement was laid in that city. As to his additional education, he studied under Mr. Easley when he was his Assistant City Engineer, and, later, took a correspondence course in Engineering, which he completed with a very high standing.

His entire professional ambitions were wrapped up in the growth of his city. During his long career of thirty-one years as City Engineer, Mr. Reynolds saw the city emerge from its first permanent sidewalk, which he constructed of brick, to the many miles of paving it now possesses. In addition, many beautiful bridges in Fort Dodge are also monuments to his activities. Another improvement which took place under his direction and of which he was exceptionally proud, was the selection of numbers for city streets in place of the names which they formerly had. This change took place about 1900.

Mr. Reynolds was probably more closely identified with the growth of Fort Dodge than any other one person. He took much pride in his native city and was always ambitious for its improvement. The last photograph he had taken was on the site of the new City Water-Works, on North First Street, where he was present to witness the turning of the first shovelful of earth.

In addition to his duties as City Engineer, he was for twelve years County Surveyor of Webster County, and for sixteen years Principal Drainage Engineer in Webster and near-by counties. He also did some general engineering practice for surrounding municipalities for sewers, bridge, and street work.

He was a member and active worker in many lodges and fraternal organizations of the city. Among them were the Elks Lodge, of which he was Past Exalted Ruler and Past District Deputy Grand Exalted Ruler. He was also very active in the Masonic Lodge, having been Past Commander of Calvary Commandery, Knights Templar, Past Patron of the Eastern Star, Past Watch-

* Memoir prepared by C. H. Currie, M. Am. Soc. C. E.

man of Shepherds in the White Shrine, and a member of Za-Ga-Zig Shrine, Des Moines, Iowa.

Mr. Reynolds was a member of the National Arbitration Commission. He was also a member of St. Mark's Protestant Episcopal Church, of Fort Dodge, and for nine years Secretary of the Oakland Cemetery Association, in which office he was responsible for the adoption of the perpetual-care plan, which has been one of the outstanding improvements in the city.

A lasting tribute to Mr. Reynolds may well be summed up in the fact that his appointment and re-appointment as City Engineer over a period of thirty-one years were never questioned; and this occurred in a city where politics and officers changed frequently.

Mr. Reynolds was very highly respected among engineers throughout the State and by every one who knew him. He was married in 1898 to Laura Ellen Beresford, of Fort Dodge. Besides his widow, he is survived by his son, Jack, and two daughters, Myriam and Helen.

Mr. Reynolds was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919.

BOYD FREEZE WALKER, Assoc. M. Am. Soc. C. E.*

Died December 16, 1929.

Boyd Freeze Walker was born in Des Moines, Iowa, on February 3, 1888. He attended the public schools and was graduated from the Des Moines High School. He then entered the Civil Engineering Department of Iowa State College, at Ames, from which he was graduated in 1912 with the degree of Bachelor of Science in Civil Engineering. He was an enthusiastic participant in athletics in High School, and remained interested in them throughout his life. He held the championship in hurdling and running at Iowa State College and also was considered one of the best "high jumpers".

Mr. Walker's first engagement was as Road Engineer for the Chicago, Rock Island and Pacific Railway Company, which position he soon resigned to go to Great Falls, Mont., where he was employed by the Ambursen Company as Construction Engineer on the Great Falls Dam. On its completion he was sent to Prince Albert, Ont., Canada, as Assistant Engineer on the construction of a large dam. From May, 1913, to May, 1914, he served as Deputy County Surveyor at Fort Benton, Mont., on office and field work on road construction.

In April, 1915, he went to Council Bluffs, Iowa, where he secured a position as Assistant to the County Engineer of Pottawattamie County, on road and bridge work. At this time he also undertook his first drainage project in this County. Mr. Walker served as County Engineer of Pottawattamie County for three years, at the expiration of which time he entered private practice, specializing in county drainage work, special investigations, and reports for corporations and individuals, etc., in his own and the surrounding counties, and establishing himself as one of the most capable engineers in that part of Iowa.

* Memoir prepared from information on file at Society Headquarters.

In the fall of 1925, a delegation of business men of Council Bluffs called on Mr. Walker asking him to consider the nomination for City Engineer. After debating the question for some time, he accepted the proposal and in March, 1926, he was elected to that office by a large majority. This position he held until his death from anemia after an illness of two months.

Mr. Walker's memory is perpetuated in Council Bluffs by many city improvements which he forwarded. He kept himself well informed on all technical and other subjects, and having been called on repeatedly to serve as Consulting Engineer by various railroad companies in their respective lawsuits, he was studying law at the time of his death.

He was a man of few words, but when he chose to speak his word was one on which to depend. He was kind to and considerate of his fellow workers and there was no undertaking too insignificant for his personal attention. He was an active worker in the various civic organizations of Council Bluffs and was always glad to give his time for any plan that would improve or better his city or State. A warm-hearted, law-abiding citizen, he occupied a most difficult position, but never at any time did he usurp his power or do that which could cause him to be criticized as unfair or unjust.

Mr. Walker was a Thirty-second Degree Mason, member of the Shrine and of the Eastern Star, as well as of the Benevolent and Protective Order of Elks and the Beta Theta Pi Fraternity. He was also affiliated with the Iowa Engineering Society, of which he was a Director at the time of his death, and the Engineers' Club of Omaha, Nebr. He was a member of the First Presbyterian Church of Council Bluffs.

On December 20, 1916, he was married to Marigold C. Robey, of Omaha, Nebr. He is survived by his widow, three children, George, Boyd, Jr., and Marigold B.; his mother, Mrs. A. H. Walker, and a brother, George L. Walker, of Des Moines; and three sisters, Mrs. J. S. Brisco, of Arroyo Grande, Calif., Mrs. D. E. Nichols, of Livingston, Mont., and Mrs. A. H. Hinkley, of Puunene, Maui, Hawaii.

As a sincere appreciation of Mr. Walker, the following may be quoted:

"A man who is faithful to family, country, God, and fellow men and efficient in the discharge of duties and obligations, is a loss to the country when he departs. The loss is greater when death comes in the prime of life as it did in the case of Mr. Walker. Not half of his normal expectancy in years had been lived, and had he been permitted he would have given good service to his country and his family and community for many years. * * * As the record stands, Mr. Walker's family and friends * * * may well feel pride in his accomplishments for they, like his character, were clean and sound and good".

Mr. Walker was elected an Associate Member of the American Society of Civil Engineers on June 10, 1929.